VOL. XXXVIII. No. 6 JUNE, 1960

THE STRUCTURAL ENGINEER

THE JOURNAL OF THE
INSTITUTION OF STRUCTURAL ENGINEERS

Tall Buildings — Problems in the Design and Construction as affecting the Structural Engineer
by Lt.-Colonel G. W. Kirkland (Vice-President)

The General Moment Distribution Analysis of Space Frames
by B. Rawlings

Uniform Load on a Slab or Plate Stiffened by Beams
by Dr. M. Holmes

A Commentary on the New British Standard 449 (1959) :
The Use of Structural Steel in Building
Discussion on the Paper by John Mason (Hon. Treasurer)

The Collapse of Triangulated Trusses by Buckling within the Plane of the Truss
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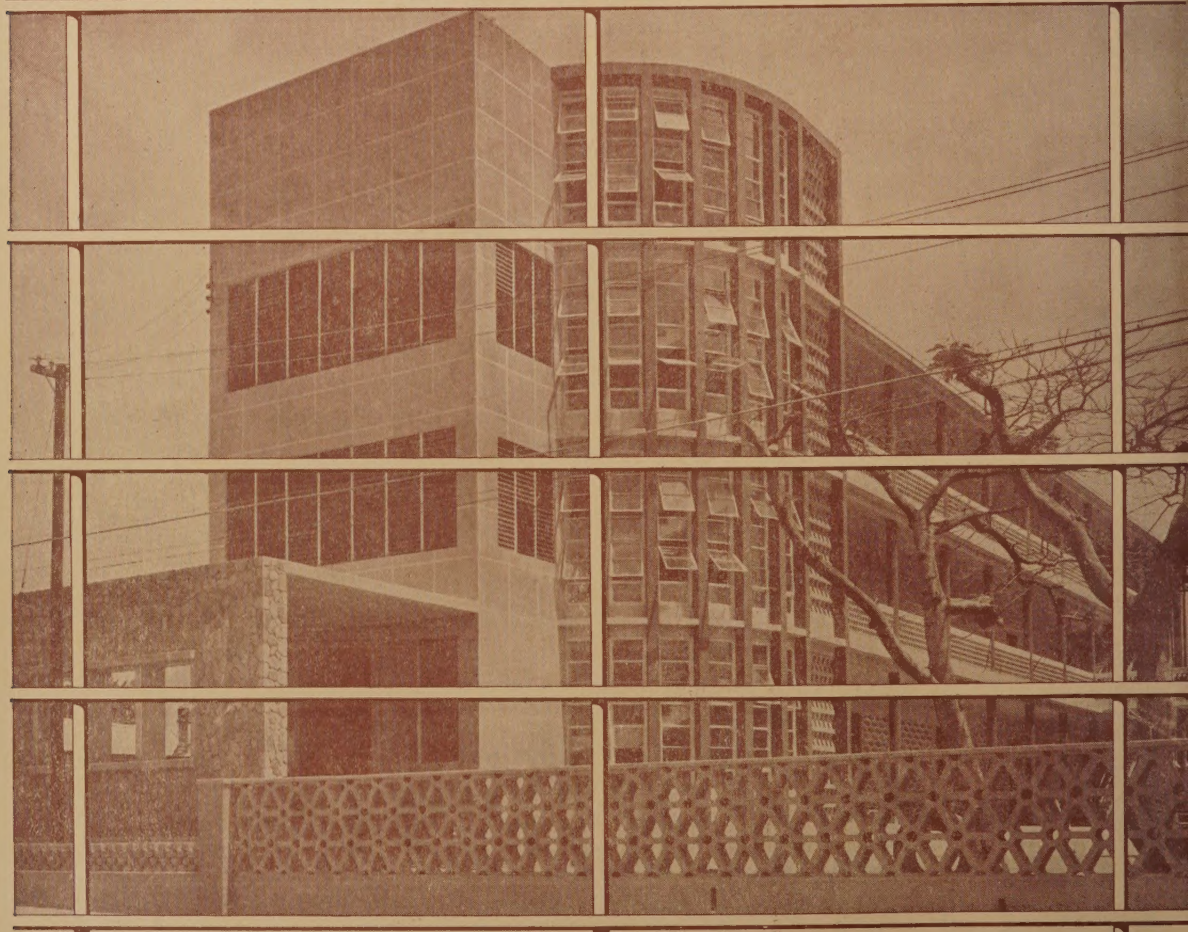

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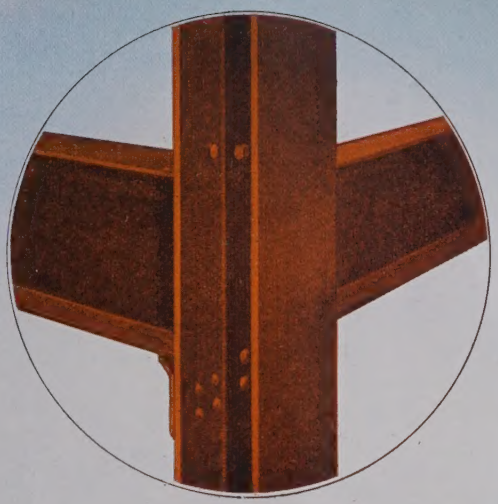

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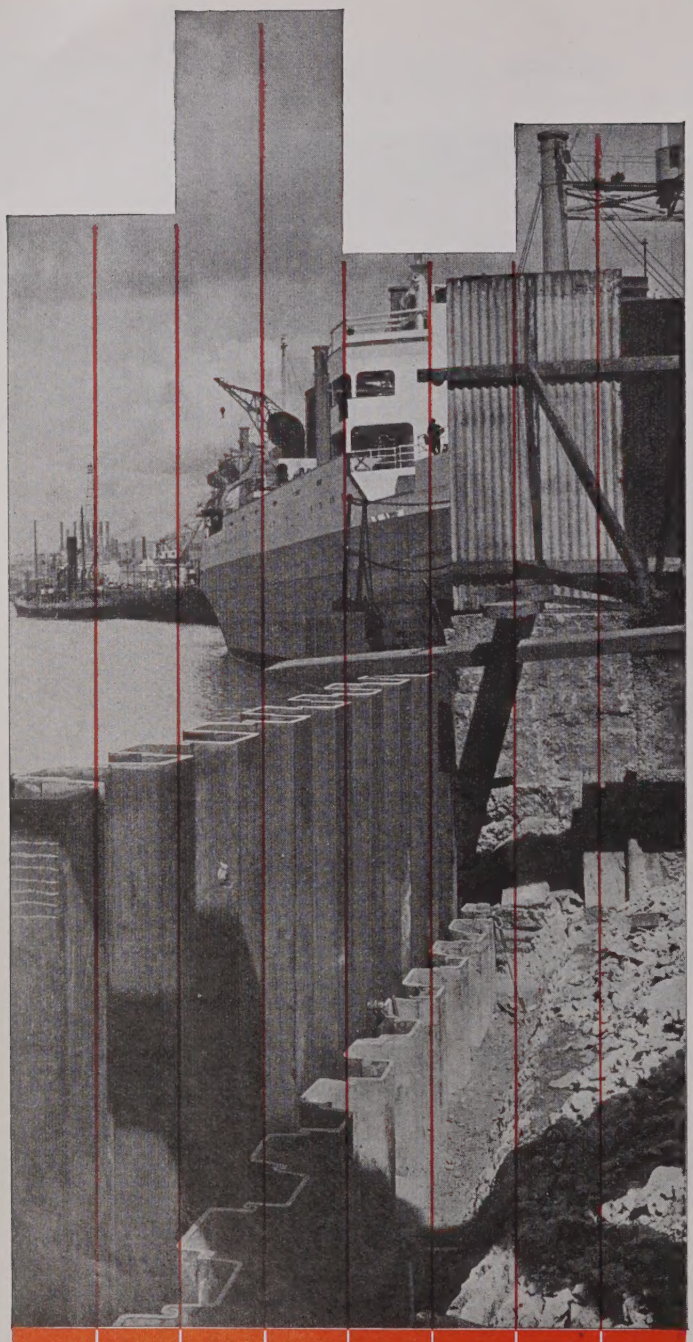

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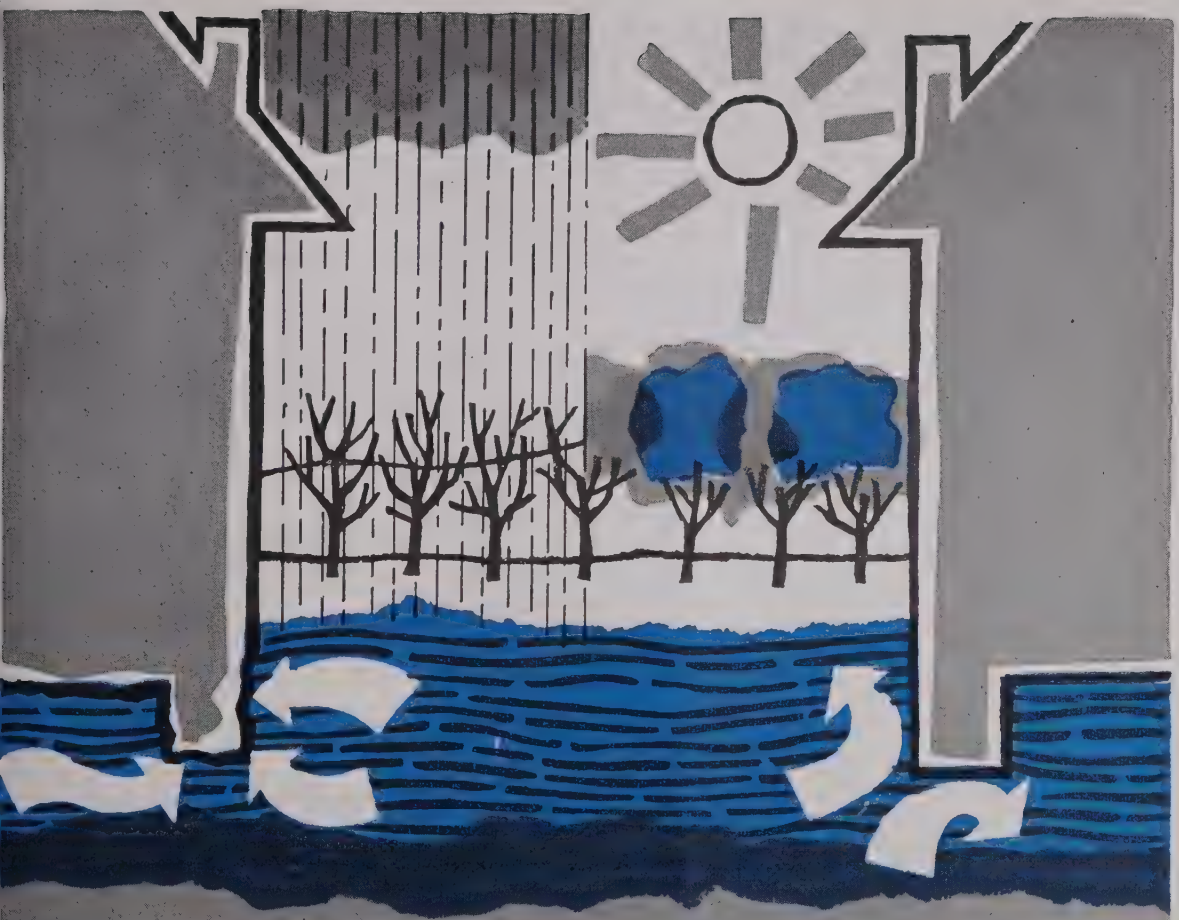

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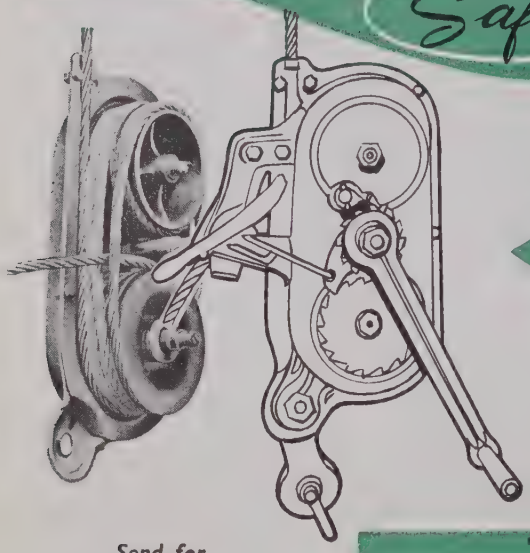

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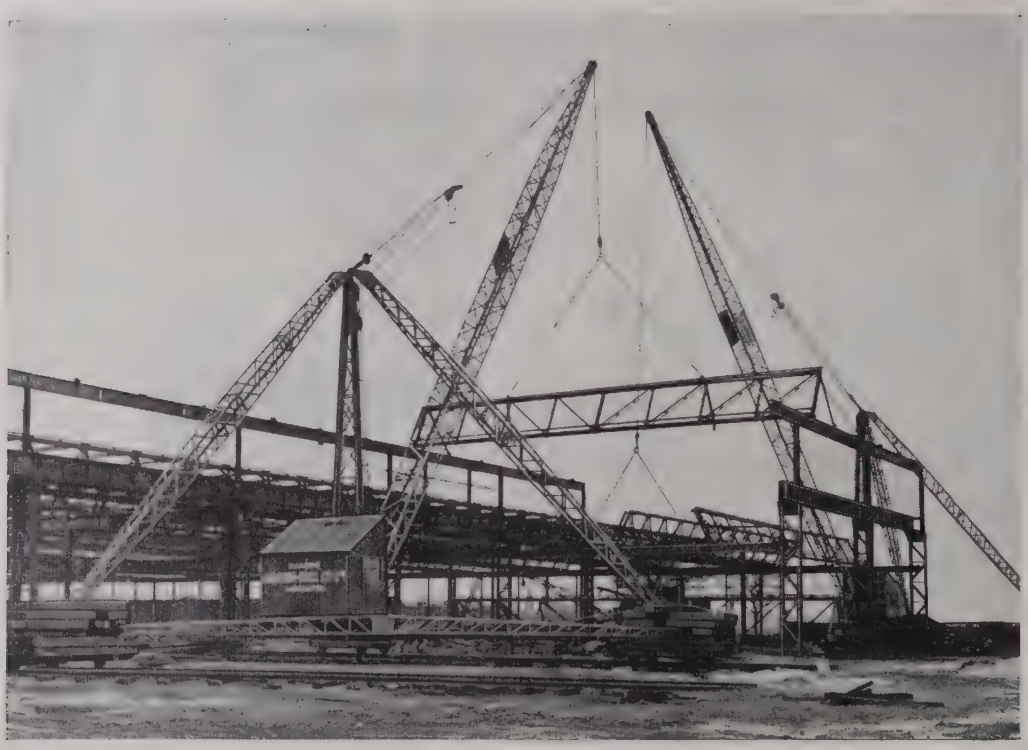

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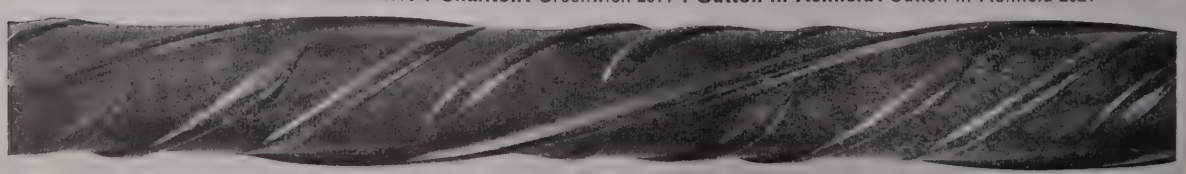

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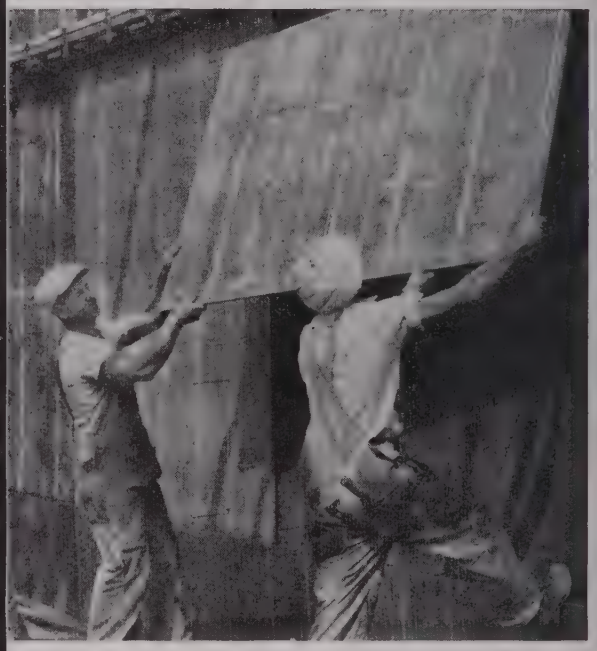

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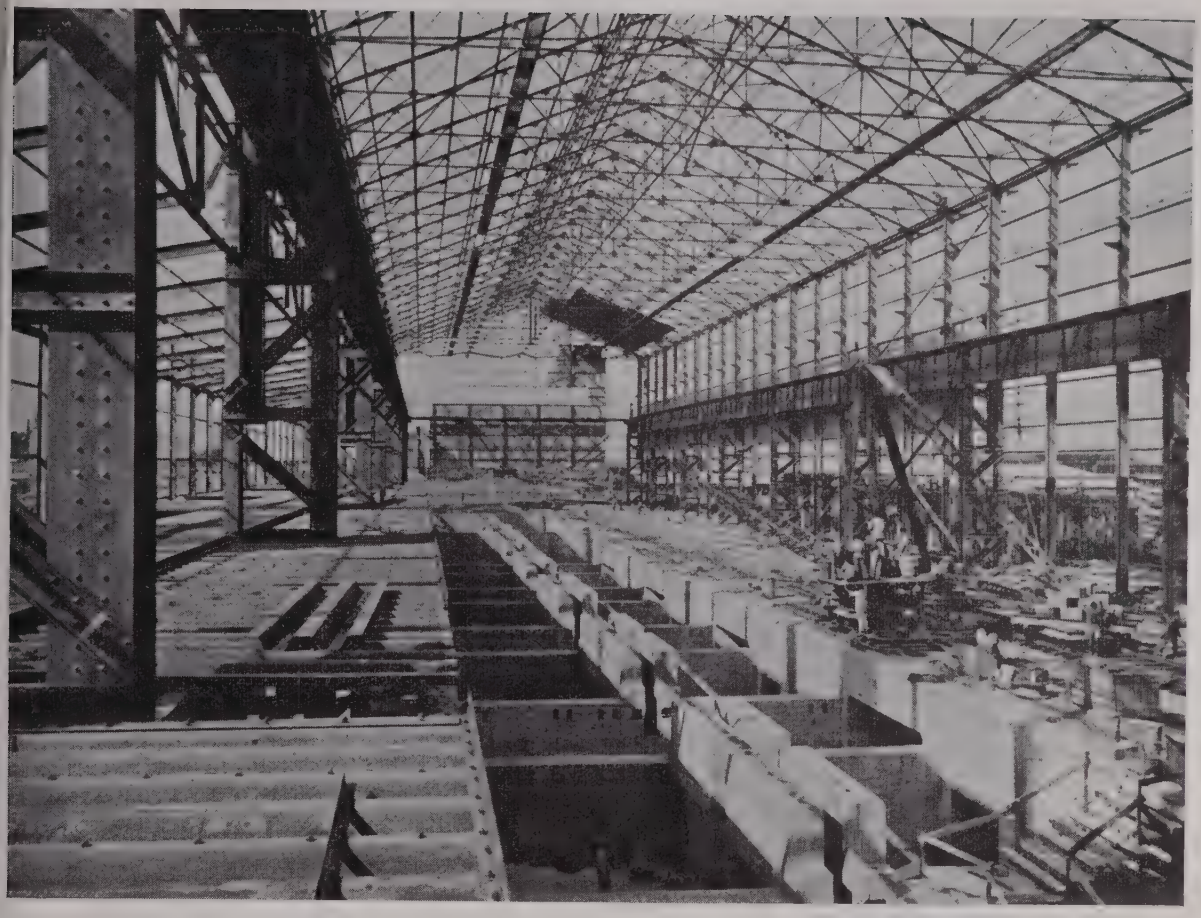

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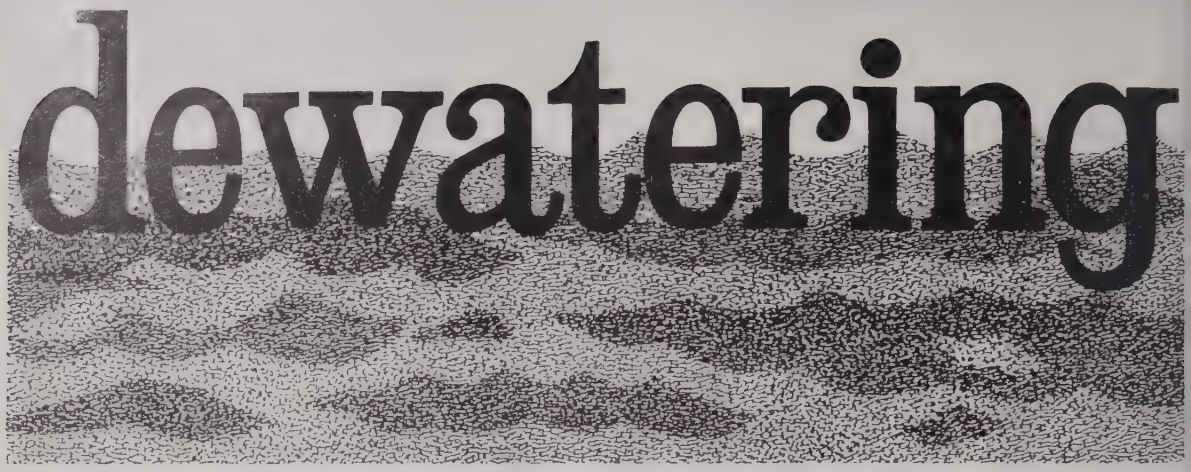

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Tall Buildings—Problems in the Design and Construction as affecting the Structural Engineer *

by

Lt.-Colonel G. W. Kirkland, M.B.E.(Mil.), M.I.Struct.E., M.I.C.E. (Vice-President).

The necessity for ever increasing elevation of buildings arises from a number of causes, and while these reasons do not form any part of the design and construction problems involved, they are, in the author's view, worth stating.

The primary reason is one of pure economics. Increasing land values, fixed plot ratios governing densities of occupation, have necessitated vertical development in order to provide the floor areas needed for an adequate return of capital invested in these modern developments.

The reduction of available site areas in comparison with areas of buildings originally occupying a site, by virtue of land surrender for street improvements, massing of small site units into a single development unit, and the limitations imposed by the Town Planning Acts, are additional factors.

It is not long ago that the eight storey building was referred to as a tall building, while today we face 11 storeys with equanimity, 20 storeys without undue comment and even 30 storeys without surprise. One of the first modern tall buildings with which the author was connected was an hotel project which was intended to rise 414 feet. Considerable design work was done on this development before it was known that planning consent could not be given for a building higher than 235 feet in the location it had been intended to build.

The author has not been surprised to read numerous recent criticisms of certain modern 'high point' developments. These were inevitable and indicate at least an appreciation that these things are now with us but, as referred to above, it is not considered that we need fear an excess number of these structures in our towns and cities. Restrictions will be inevitable, since any major development is equivalent to the introduction of a small township in terms of numbers onto a restricted site. This expansion of local population introduces numerous secondary problems which are of major importance, not the least of which are transportation, water supplies and sewage and waste disposal.

That we shall continue to build high is inevitable and it is with the object of providing a forum for the exchange of knowledge, information and ideas on the subject of structural design and site construction problems that this paper is now presented.

The design problems cannot be dealt with under one general head. The high point building presents different problems to those involved in the massed building of considerable elevation.

All the problems are not new ones, and it may be of interest to know that almost 30 years ago the author was associated with the design and construction of a 220 feet high refinery building for Tate & Lyle Ltd. at Plaistow Wharf Refinery (Figs. 1, 2 and 3). The Embankment front of Shell Mex House, on the site of the old Hotel Cecil, another building completed a quarter of a century ago and in connection with which the author was a member of the design team, rises to no mean elevation and was, at the time of its construction, regarded as the maximum height possible. During this same period, the author was privileged to examine and check the wind calculations for the Empire State Building in New York.

Even amongst the partially informed of those days it was believed that the limit of construction height had been reached for London, firstly, because of the height limit of fire brigade apparatus and, secondly, because of the inability of London blue clay to support the loads which would arise from higher structures.

On securing an appointment as Consultant for a very tall building in London, advantage was taken during a visit to the U.S.A. to examine a number of 'skyscrapers' under construction. As a result of these visits it was found that methods of construction had so profound an influence on structural design, that a knowledge of, availability of, and experience in the use of specialised plant was an essential in Contractor selection. The team involved in the new Millbank Development has had the good fortune to have included in it the Contractor, and the view is expressed that such an arrangement leads to the best in economy, design and construction. On a development where an overall saving of two months in completion time can represent a saving of £150,000 in terms of rent for occupiable space, interest on capital, etc., it will readily be appreciated that actual costs of the pure construction need not necessarily be critical. In advocating the earliest possible inclusion of the Contractor in the team, the author would like to make it perfectly clear that he does not support in any way the procedure of the Contractor/Designer. The growing tendency today for the larger contracting firms to employ as members of their staff, architects and design engineers, is not in the best interests of either the public or the industry. The claim so frequently made that such an arrangement saves fees is completely without foundation since no contractor gives anything away if he is going to keep his shareholders happy, and the alternative of ignoring profits on construction in favour of the tax relieved capital appreciation within a controlling development company is but a snare and delusion, in that the

* *Read before The Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1. on 7th April 1960.*

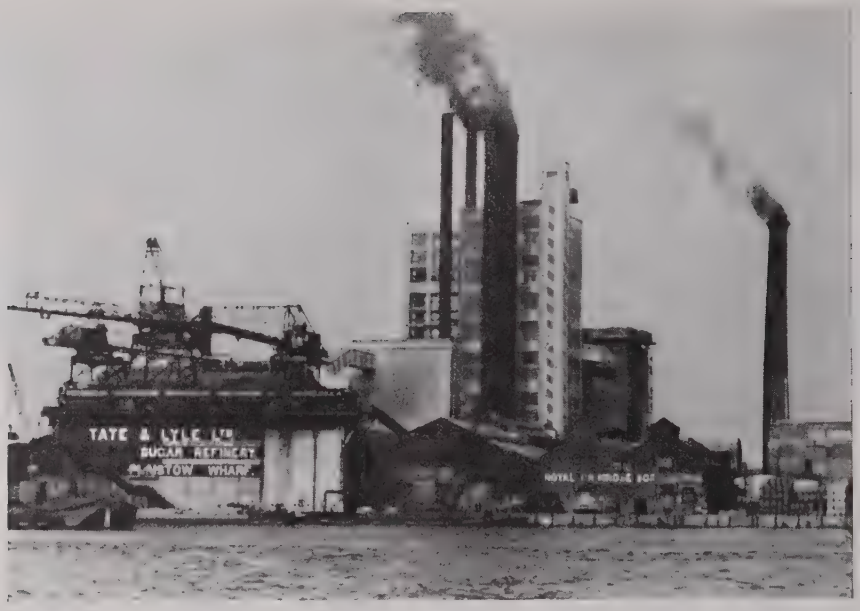

Fig. 1.—View of the 220 ft. high Refinery Building constructed in 1935 for Tate & Lyle Ltd. at Plaistow Wharf.

ultimate building owner or user still pays. The author's comments are unlikely to concern the large financial interests involved, but he felt the point had to be made. So much for economics.

The use of any floor area may be closely allied to the amount of light the space receives naturally, the readiness and speed of access, and the efficiency of the sundry ancillary services. A tall building therefore, must be well equipped with lifts, and large duct areas to provide for vertical distribution of personnel and services. In the case of the high point development, the massing of lift lobbies, stairs and ducts in a convenient central area provides a central core of which structural use can be made by making such a core sufficiently stiff to resist the whole of the not inconsiderable wind forces. In cases of other than high buildings, lift lobbies and stairs are normally required to be distributed on the plan. In these cases the walls can be developed as strong points of as great width as possible and can be used as buttresses against the horizontal forces involved. The value of solid gable walls is frequently reduced by the foundation problem when the building abuts another site. In both cases, wind loads are distributed by the cladding onto the columns which act as beams in the vertical plane, and transmit the loads through the floor slab on to the stiff core or abutments. The normal floor slab is quite capable of functioning as a horizontal girder provided adequate attention is paid to the edge conditions.

While dealing with the subject of wind loading it is advisable to mention the little appreciated ' vortex ' condition which arises with certain building shapes. In addition to normal wind loading there sometimes builds up on the leeward side of a structure, a vacuum as the result of the vortex created by the wind. This vacuum does not remain constant but increases in volume until equilibrium breaks down and the vacuum ' bleeps ' away into the atmosphere. The rate of ' vacuum bleep,' or vortex shedding, varies with the wind velocity and critical periods can occur with wind velocities as low as 10 to 15 m.p.h. In the case of certain modern claddings, the incorporation of projecting mullions varies the condition by the creation of secondary vortices leeward of each projection. As these smaller vortices are shed the equilibrium of the major vortex is disturbed and the rate of discharge is affected. Mathematical analyses of natural frequencies of structures are possible, as are approximations of the rate of vortex discharge and such preliminary examination should enable the designer to determine whether the condition is likely to be critical or not. In case of doubt, wind tunnel tests with a model are advisable.

The tall structure with wings may be deficient in torsional stiffness, and this condition can arise from an eccentricity of plan, or an off centre strong point. In a number of new developments on the Continent and at home, engineers have carried out wind tunnel tests to satisfy themselves on this issue.

In Britain, the maximum height limitations imposed necessitate minimum storey heights to make a paying proposition of many developments. In the U.S.A. no such restrictions apply and it becomes possible and frequently desirable to provide deep floor spaces for distribution of the various services. These ' voids ' allow of deep floor supporting members and make it possible to use structural steel with heavy end connections to cope with wind conditions.

The use of reinforced concrete has many advantages in providing the minimum storey heights advisable in this country and economies arise from better end fixing conditions, the homogeneity of beam and slab, not to mention the advantages of the flat slab or plate floor where possible.

The facility of placing concrete in masses of a weight within the hoisting capacity of the site equipment and the use of composite in-situ and precast construction are assets of which both contractor and designer can take advantage.

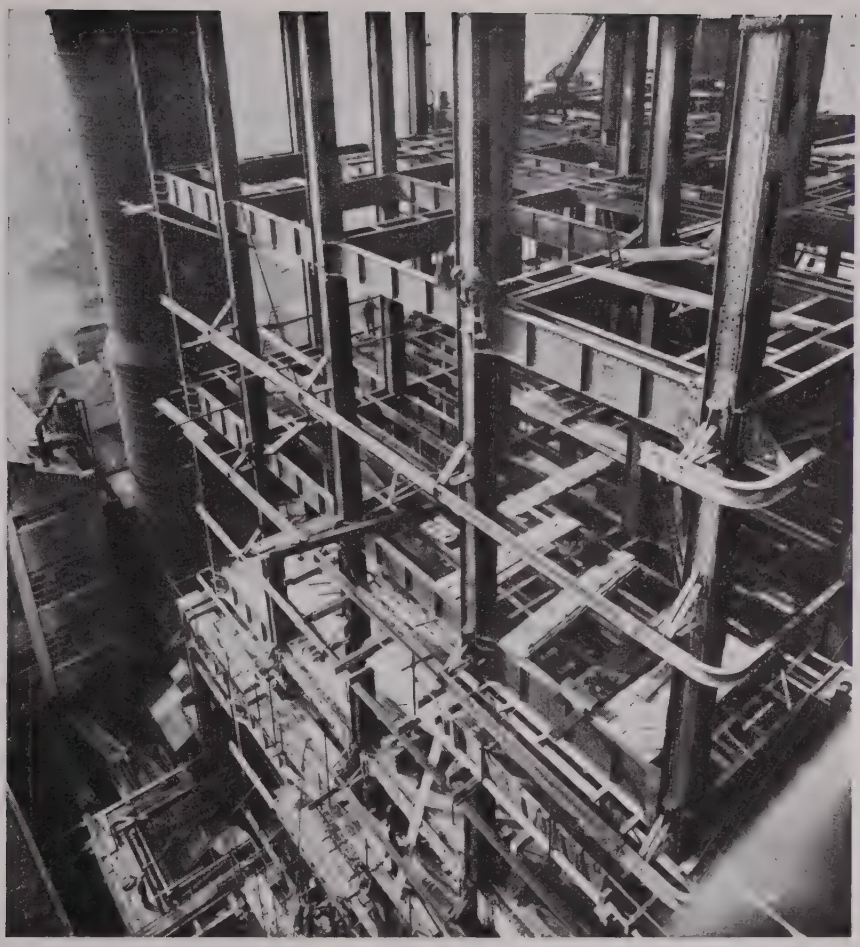

Fig. 2.—Progress Photograph of the steel structure to the Plaistow Wharf Refinery.

Note outrigging members to carry one of the earliest forms of curtain wall.

It has been proved that the precision of concrete construction can equal that of any other building medium, and strength controlled concrete, mixed to a designed ratio of ingredients, gives the designer considerable freedom in meeting the structural requirements.

On the construction side, the costs of handling sundry building components and materials are of major importance and while prefabrication has many advantages in either steel or concrete, the designer must be aware of the construction problems arising from heavy lifts of material and prefabricated parts, also the type of temporary load condition likely to be imposed on the structure by the contractor's plant performing the essential hoisting operation. A steel stanchion to carry 1,000 tons will weigh almost a quarter of a ton per foot run, a 12 feet storey height 3 tons, and three such storey heights 9 tons. The lifting problems involved in elevating these and comparable masses by stick or derrick supported on an incomplete structural frame and ensuring that the structure remains plumb, necessitates rigidity of connections at each stage of the work and the provision of guys and temporary bracings. Loadings induced by these conditions must be appreciated at the design stage. This is, in the author's view, yet a further indication of the advisability of closest Designer/Contractor liaison from the earliest possible stage in the development. No two contractors are likely to offer the same solution to any constructional problem, and each alternative solution can cause major variations in design.

As yet, the subject of services has been barely touched in this paper, but they are major headaches for both Architect and Engineer. In building tall, the structural requirements are of even greater importance than in buildings of conventional elevation, and the Architect often must needs subjugate his conceptions where an alternative is possible to the structural necessities. Of what advantage is it to either Architect or Engineer, however, if in producing the idealised structure, the services, which are essential to make the structure habitable, are inefficient or given only second place to other interests? The very tall building may well be likened to an ocean liner so far as its services are concerned, and a like degree of co-operation between the participants in its construction is needed to create the successful tall building, as with the successful liner.

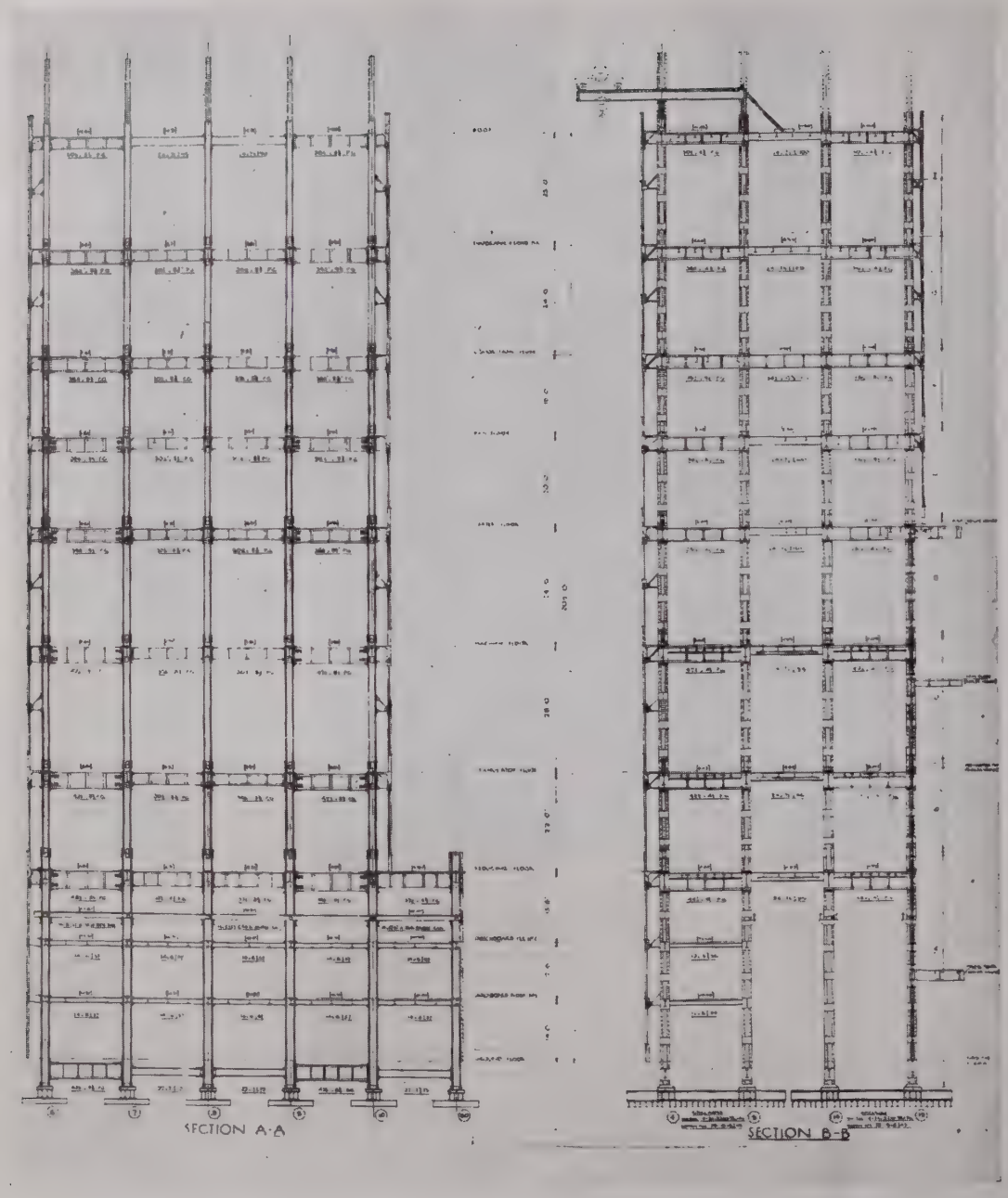

Fig. 3.—Structural cross section through the Plaistow Wharf Refinery.

We permit much self-criticism in Britain when comparing rates of construction here with those in many other countries, particularly the U.S.A. Much of this is not justified. The author is convinced of the excellence of the quality of our larger (and many smaller) building and engineering contractors, and of our designers, both architectural and engineering, else how could our building proceed at all under conditions which necessitate the contractor starting work before he has the whole of the information required to complete? In a number of developments investigated in New York and Chicago, some two to three years' work in planning design and detail had been completed before even demolition works on the site had commenced. The contractor was then in possession of wellnigh the whole of the construction information, down to size of small holes in structural members, and half inch details of door architraves before work

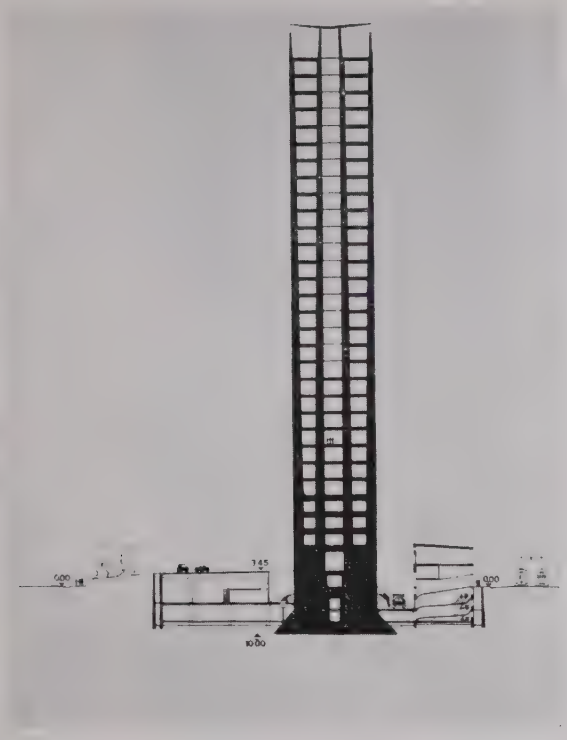

Fig. 4.—Transverse section through the Pirelli Building, Milan.

commenced. Under these conditions, construction work is carried out almost as a military operation and speed of construction can be accurately planned. This ideal is worth striving for, though employment of the necessary building finance may well be a strong negating factor.

While some of the complexities of superstructure design have been mentioned, in many parts of Britain and certainly throughout the London area, the foundation problem is of even greater importance.

Engineers are aware of the sometimes necessarily complex foundation solutions necessary for buildings of only conventional height, and the tall building increases the magnitude of this problem. London blue clay, capable of normal bearing values of from $2\frac{1}{4}$ to $2\frac{3}{4}$ tons per square foot is required to support structures which, on a basis of total load carried to net area of the ground covered, would produce comparative pressures of from six to as much as ten tons per square foot.

Only recently we read a Parliamentary statement that 'Big Ben' tower was out of plumb and that movement had occurred in the Victoria Tower. While horizontal movements of 15 inches were spoken of, the vertical movement cannot but be sufficient to create a problem in the maintenance of floor levels.

The tower of Westminster Cathedral has settled substantially and visibly, but these settlements in themselves do not cause undue alarm. The problem arises from the differential settlement between component parts of the building mass.

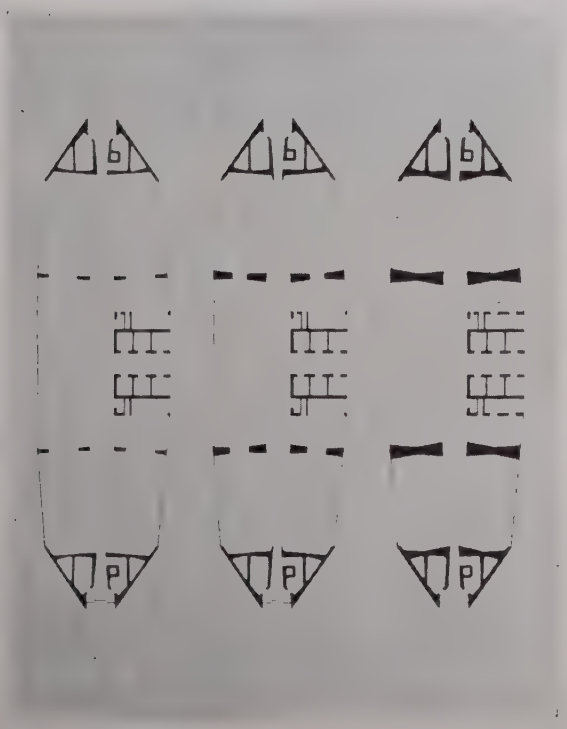

Fig. 5.—Floor Plans of the Pirelli Building, Milan, illustrating stiffened cross walls.

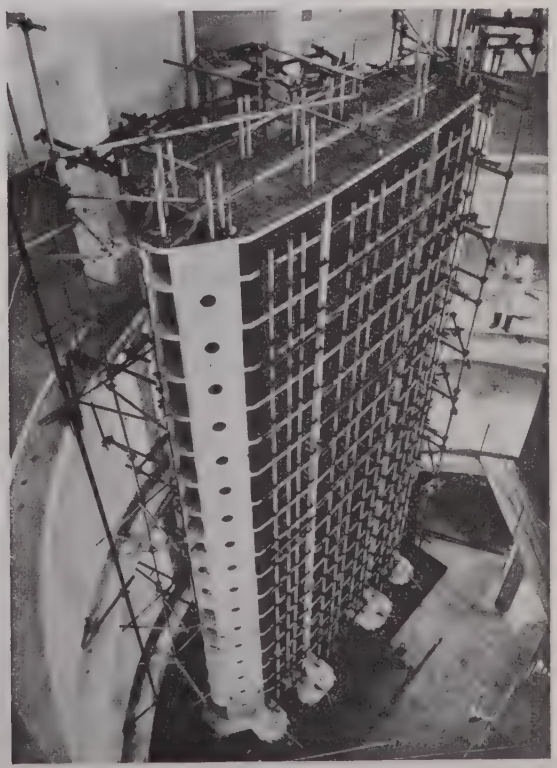

Fig. 6.—Model of the Pirelli Building, Milan, (Scale 1/5th), undergoing test in the ISMES laboratory.

Fig. 7.—Completed Pirelli Building.
(*Photograph by courte y of S.p.a. Bologna*)

Fig. 8.—Building No. E.100 for the BASF at Ludwigshaven under construction.

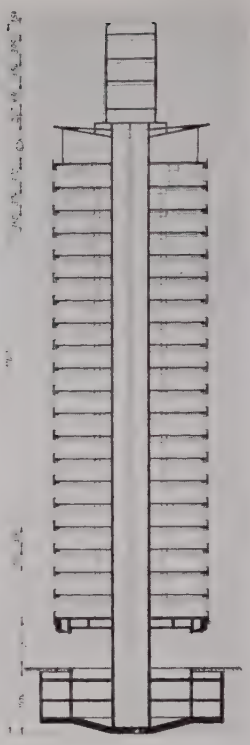

Fig. 9.—Cross section through Building No. E.100 for the BASF at Ludwigshaven.

There have recently been valuable developments in plant which produce large diameter shaft piles and these, in the author's view, probably offer the greatest aid to solution of the foundation problem. Some foundation firms advocate the use of an expanded base as large as 10 feet diameter at the base of such piles, but extensive tests carried out under the author's control in London clay do not appear to substantiate claims made for the bell ended shaft pile. The large diameter plain shaft pile, on the other hand, has proved capable of carrying loads based on conventional analysis where the largest part of the load is absorbed by the skin friction on the shaft. It appeared from the tests referred to that the effect of the bell was to reduce substantially the resistance from skin friction and to transfer much of the load to the base of the bell. The high intensity of stress at this position gives rise to large settlements. The plain pile, gaining support with each foot it penetrates load bearing strata, has a greater load spread in the clays and, at the same depth as the bell ended pile would be founded, has transferred the load of the supported structure over a larger total area, thus giving smaller settlements and utilising a greater volume of the supporting media. In the December, 1959, number of 'Géotechnique,' Professor A. W. Skempton has written a paper on 'Cast in-situ Bored Piles in London Clay' which reports in some detail part of the tests referred to.

The bell ended pile does have value, however, in special cases, where, for example, it is advisable to avoid sewers, tunnels and the like, but the design must be based primarily on relative settlements if structural distortion is to be avoided.

Fig. 10.—Model of the Millbank Development now under construction.

The science of ' Soil Mechanics,' so long the Cinderella, now comes into its own. Forecasting of settlements by virtue of the knowledge of soil characteristics is becoming almost as exact a process as the estimation of structural stresses, and the value of the specialist in this field is more and more appreciated.

In the development involving the very tall building abutting or linked with buildings of lesser height, the solution of the problem of differential settlements needs the fullest understanding of the Architect, and his appreciation of the advice given by the Engineer.

The incorporation of straight joints, hinges, etc., in the structure, flexible links to the drainage system and the treatment of continuous faces over the areas of any link between elements liable to be subject to different rates of movement, are all points to be carefully considered.

If common floor levels are to be obtained on completion of construction, provision must be made for the initial construction settlements, and these may be estimated from the data provided by pile test loading. Provision of a construction settlement of 1 inch has been made for the Tower Block at Millbank, while the adjoining two storey Podium has no such provision, it being considered that settlements here will be negligible.

The distribution of the loads from the tall building structure onto the piles presents a further interesting problem. The minimum centres of the piles, with the safe carrying capacity per pile, determine the load which can be carried over a specific area, and will determine the maximum possible weight which may be supported.

The pile cap design will be found to resolve itself into a complex raft capable of transferring the loads to be carried over the area of the building. The conventional bearing values for London blue clays to which we are accustomed to work vary from say $2\frac{1}{2}$ tons to $3\frac{1}{2}$ tons per square foot depending on the shear value of the clay and the settlement envisaged. The Tower Block of the Millbank Development imposes a load of the order of 5 tons per square foot over the

area of the structure, without taking into account the effect of wind loads.

Engineers are apt to regard the effect of wind loading so far as foundations are concerned with some element of doubt. A few years ago, the author was concerned with an underpinning operation and decided to use piles with a jacking device making use of the dead weight of the structure. He was particularly interested to observe the jack pressure variations which occurred when nearing the marginal conditions, and subsequent investigation indicated that these additionally induced pressures resulted from a wind loading condition—despite the fact that the structure involved was within the Code of Practice limits for wind stress design. Resulting from these early observations, the matter was pursued further.

The effect of steady wind loading was apparently negligible due to the inertia of the complete structure and the elasticity of the frame, yet under gust load conditions an appreciable difference in loading was observable.

As with so many practical problems the Engineer must handle, it was not permissible to investigate the phenomenon thoroughly, but it was apparent that there was a measurable time lag between gust load condition and foundation resistances.

Advantage has been taken of this condition in catering for the wind loading on the Millbank Tower Block. The raft pile cap which rests on a good ballast bed overlying the blue clay has been made to absorb the difference between dead plus superimposed load and wind loading.

While this decision may appear to have been an arbitrary one, it does not seem unreasonable, since the ballast overlying the clay must be in a state of initial compression when the piles come under load. The additional pressures due to wind load are well within the safe maxima.

Finally, the problem of waterproofing of subground works needs consideration. The author is of the opinion that an asphalte membrane for tall structures is not a sound solution. Any membrane on the external face of subground works must be incomplete if continuity is to be obtained between substructure and superstructure.

Further, the types of foundation which are likely to be used in the United Kingdom, except for the few places where hard rock can be found at a reasonable depth, will inevitably be subject to elastic deformation, and evidence would appear to throw doubt on the ability of mastic asphaltes to cope with these movements over the entire period of the life of a structure. It may well be that the use of resinous plastics of the Vynil or Polythene type may ultimately be found suitable, but the author pins his faith in sound dense concrete cast in such lengths and masses that crack control can be reasonably assured, and in the use of a water bar at those planes of weakness to moisture entry, the construction joints.

It has only been possible to touch on a few of the problems with which the Structural Engineer finds himself involved. Amongst others are the structural fire resistance, methods of securing services to the structure, and the types of finishes to be recommended, both horizontally and vertically, on an elastic structure.

In conclusion, the author would like to thank Tate & Lyle Ltd., especially Mr. Carmichael, for the provision of photographs and leave to publish them; Dorman Long & Co. Ltd., and Mr. J. W. Anderton, A.M.I.Struct.E., for structural drawings and details so freely obtained; Messrs. Ronald Ward & Partners and Vickers Ltd., for permission to use illustrations of the Millbank Development; also the Cement and Concrete Association for their ready assistance with photographs and slides of Continental works.

Book Review

Méthodes Générales d'Essai et de Controle en Laboratoire. Livre I : Mesures Géometriques et Mécaniques. By R. L'Hermite (Paris : Eyrolles, 1959). 6 in. x 10 in., 742 pp., 475 figures.

This treatise has been specially written for anyone interested in expert opinions based upon tests of the materials to be used in constructional work. It is thorough and very well presented, and may well be recognized of vital importance to all Engineers, Contractors and others concerned with tests on materials and structures who need to keep up to date with the latest methods available for giving the most accurate results. It enables the Designer to specify accurately the materials which are most suitable for the structure under consideration.

This book is the first of a series, and deals chiefly with measurements of length, areas, forms, deformations, porosity, mass, forces, time, velocities, and supplies a most useful section devoted to the machines and apparatus used for testing. There is also an interesting chapter on the organization and layout of laboratories with a list of the most important ones throughout the world, with particular emphasis on mobile laboratories for soil mechanics tests.

As a preview, the Author gives a list of the subjects included in his second volume which refers to the measurements of temperature, humidity, viscosity together with optical and electrical computations. Later volumes in the series will deal with failures and accidents and means to avoid further occurrences.

The equipment described in the present treatise includes machines for compression, bending, torsion, fatigue, abrasion and penetration tests, and also triaxial and dynamic tests. The chapters dealing with various types of dynamometers, extensometers and apparatus for measuring double curvatures are of special interest. P.G.

The General Moment Distribution Analysis of Space Frames

by B. Rawlings, B.Sc., M.Eng.Sc., B.E., A.M.I.E. Aust.

Synopsis

A general elastic moment distribution method is presented, capable of application to all types of rigid space frames. The method employs matrices of stiffness and ' carry-over ' terms, and the procedure is equivalent to the well known Hardy Cross process of analysis. The method is illustrated by typical problems.

I. Introduction

The elastic moment distribution method of analysis of plane framed structures, developed by Hardy Cross[1] about thirty years ago is very well known to-day, being one of the foremost tools of the structural engineer who is concerned with rigid frame design. Little has been published, however, on the analysis of space frames, especially on those cases involving members oblique to the plane in which the moments are being examined. In the case of orthogonal frames the procedure is simple as plane analyses may still be performed with simple extensions to allow for the torsional resistance of those members normal to the plane[2]. However, when the angle between such members and the moment plane is acute, deformation occurs partly as torsional deformation about the longitudinal axes of the members, and partly as flexural deformation normal to these axes. In addition, because of interaction, it becomes impracticable to reduce the procedure to three plane analyses. As rigid space frames are becoming more popular, it is evident that a general analytical procedure having the advantages of the moment distribution method would be very desirable. It is thus the aim of this paper to extend the moment distribution analysis to include space frames and to apply the theory to typical examples. The following treatment is restricted to the case of frames of uniform members, but could be modified to allow for members of variable moment of inertia. The method has the additional advantage that it may also be extended to cover the case of space frames with semi-rigid joints, the analysis of which would otherwise be extremely arduous. In common with the plane moment distribution procedure it is assumed that, in the case of unbraced frames, deformation occurs primarily from flexural (and torsional) action, and in triangulated rigid frames that the effect of axial forces in members is predominant. Thus, in the latter case, if the shears accompanying the secondary moments are large, it may be necessary to make corrections to the axial forces and repeat the distribution process. Even in such a problem the method may prove to be considerably more expedient than a full strain energy or relaxation analysis[3,4,5]. It is convenient, in developing the theory to examine firstly the case of a single member, when subjected to moments and torsion at the ends.

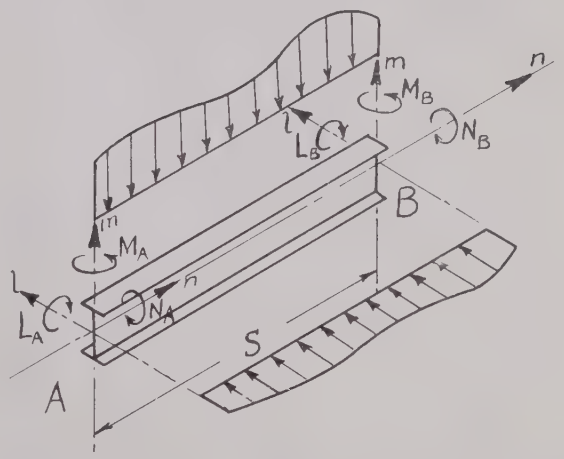

Fig. 1

II. Relationship Between Moments and Deformation of a Single Member

Consider a member AB, having a length S and symmetrical about both its major and minor cross-sectional axes as shown in Fig. 1. This member is subjected to moments L_A, M_A and N_A at the end A and to L_B, M_B and N_B at the end B, the axes of these moments being respectively the two principal axes and longitudinal axis of the member, positive directions being selected to give right-handed sets. The member is also subjected to transverse loading, the components of fixing moments at the ends A and B being $L_{(F)A}$ and $L_{(F)B}$ about the l axis, and $M_{(F)A}$ and $M_{(F)B}$ about the m axis. Positive moment will be considered to be that obeying the right-hand screw rule.

It is now possible to relate the deformations of the member to the applied moments. Consider firstly the flexural deformation caused about the major axis l by the end moments L_A and L_B. From slope-deflection analysis it can be shown[6] that

$$L_A = \frac{2EI_l}{S}\left(2\varnothing\, \iota_A + \varnothing \iota_B + \frac{3\delta_{mB}}{S}\right) + L_{(F)A}$$

$$= 2B_l\left(2\varnothing\, \iota_A + \varnothing \iota_B + \frac{3\delta_{mB}}{S}\right) + L_{(F)A} \quad \cdot \quad (1)$$

where I_l is the moment of inertia of the section about the l axis, $\varnothing \iota_A$ and $\varnothing \iota_B$ are the rotations of the ends A and B about the respective l axes, considered clockwise in conformity with the right-hand screw rule, and δ_{mB} is the displacement of the end B in the positive m direction.

Similarly

$$L_B = 2B_l\left(\varnothing l_A + 2\varnothing l_B + \frac{3\delta_{mB}}{S}\right) + L_{(F)B} \quad \cdot \quad (2)$$

Secondly, by examining the flexural deformations occurring from moments applied about the minor axis m, the following equations are obtained in a similar way.

$$M_A = \frac{2EI_m}{S}\left(2\varnothing_{mA} + \varnothing_{mB} - \frac{3\delta_{lB}}{S}\right) + M_{(F)A}$$

$$= 2B_m\left(2\varnothing_{mA} + \varnothing_{mB} - \frac{3\delta_{lB}}{S}\right) + M_{(F)A} \cdot (3)$$

and

$$M_B = 2B_m\left(\varnothing_{mA} + 2\varnothing_{mB} - \frac{3\delta_{lB}}{S}\right) + M_{(F)B} \cdot (4)$$

Finally, the torsional moments N_A and N_B, necessarily equal and opposite for equilibrium of the member, will cause the ends to twist relative to each other in accordance with the relationships

$$N_A = \frac{C}{S}(\varnothing_{nA} - \varnothing_{nB})$$

$$= B_n(\varnothing_{nA} - \varnothing_{nB}) \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (5)$$

and

$$N_B = -B_n(\varnothing_{nA} - \varnothing_{nB}) \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (6)$$

where C is the torsional rigidity of the member. This will be considered to have been established for similar conditions of warping restraint to that anticipated when the member constitutes part of the frame.

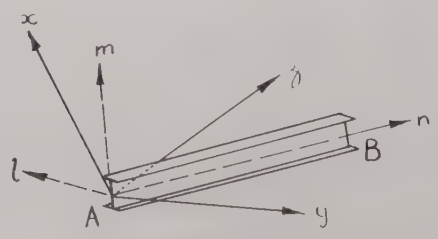

Fig. 2

This typical member will, in general, be at some random orientation to a new standard right-hand orthogonal set of axes (x, y, z), to which all rotations and moments in the space frame will be referred. Having established the relationships between the applied moments and the deformations of the member about the (l, m, n) set of axes, it thus becomes necessary to relate these to the new (x, y, z) set, as shown in Fig. 2. In order to do this, it is first necessary to relate the inclinations of the axes by some convenient means. This can be done in various ways, but one which has many advantages is the use of direction cosines. These are the cosines of the nine angles subtended between an axis in one set and an axis in the other set, both being drawn in the positive direction. It will be seen that as each angle may have a value between 0 and 180° due regard must be paid to the sign of each direction cosine. For example, the angle lAy (Fig. 2) is greater than 90°, so that the cosine, denoted by $\widehat{yl}$ would be negative for this particular case.

The set of nine direction cosines may readily be written in the array

$$\begin{bmatrix} \widehat{xl} & \widehat{yl} & \widehat{zl} \\ \widehat{xm} & \widehat{ym} & \widehat{zm} \\ \widehat{xn} & \widehat{yn} & \widehat{zn} \end{bmatrix}$$

to form the square orthogonal matrix (**M**) which has the special and convenient property that its inverse is equal to the transposed matrix in which the columns and rows are interchanged.

Thus

$$\mathbf{M}^{-1} = \mathbf{M}^{T} = \begin{bmatrix} \widehat{xl} & \widehat{xm} & \widehat{xn} \\ \widehat{yl} & \widehat{ym} & \widehat{yn} \\ \widehat{zl} & \widehat{zm} & \widehat{zn} \end{bmatrix}$$

(It should be noted here that in general the numerical procedure for finding the reciprocal of a 3 × 3 matrix will be described later in the paper).

Points referred to one set of axes may be referred to the other set by means of the matrix equations,

$$\begin{bmatrix} l \\ m \\ n \end{bmatrix} = \begin{bmatrix} \mathbf{M} \end{bmatrix} \begin{bmatrix} x \\ y \\ z \end{bmatrix} \quad \cdot \quad \cdot \quad \cdot \quad (7)$$

or

$$\begin{bmatrix} x \\ y \\ z \end{bmatrix} = \begin{bmatrix} \mathbf{M} \end{bmatrix}^{-1} \begin{bmatrix} l \\ m \\ n \end{bmatrix} \quad \cdot \quad \cdot \quad \cdot \quad (8)$$

Resolving the moments L, M and N into three components X, Y and Z with vectors in the positive x, y, z direction, one obtains

$$L = X\cdot\widehat{xl} + Y\cdot\widehat{yl} + Z\cdot\widehat{zl}$$

$$M = X\cdot\widehat{xm} + Y\cdot\widehat{ym} + Z\cdot\widehat{zm}$$

and $$N = X\cdot\widehat{xn} + Y\cdot\widehat{yn} + Z\cdot\widehat{zn}$$

which may be expressed in matrix form as

$$\begin{bmatrix} L \\ M \\ N \end{bmatrix} = \begin{bmatrix} \mathbf{M} \end{bmatrix} \begin{bmatrix} X \\ Y \\ Z \end{bmatrix} \quad \cdot \quad \cdot \quad (9)$$

and which, upon inversion of equation (9) becomes

$$\begin{bmatrix} X \\ Y \\ Z \end{bmatrix} = \begin{bmatrix} \mathbf{M} \end{bmatrix}^{-1} \begin{bmatrix} L \\ M \\ N \end{bmatrix} \quad \cdot \quad \cdot \quad (10)$$

Also, it is required to relate the angle changes in the two frames of reference. It will be assumed here, as in all small deformation theory, that the angles are small (i.e. their radian, sine and tangent measures are sensibly equal). Under these conditions the angle changes referred to the (l, m, n) set will transform to angle changes (denoted by θ) referred to the (x, y, z) set by the matrix equation

$$\begin{bmatrix} \varnothing_l \\ \varnothing_m \\ \varnothing_n \end{bmatrix} = \begin{bmatrix} \mathbf{M} \end{bmatrix} \begin{bmatrix} \theta_x \\ \theta_y \\ \theta_z \end{bmatrix} \cdot \quad \cdot \quad \cdot \quad \cdot \quad (11)$$

which inverts to

$$\begin{bmatrix} \theta_x \\ \theta_y \\ \theta_z \end{bmatrix} = \begin{bmatrix} \mathbf{M} \end{bmatrix}^{-1} \begin{bmatrix} \varnothing_l \\ \varnothing_m \\ \varnothing_n \end{bmatrix} \quad \cdot \quad \cdot \quad \cdot \quad (12)$$

Finally, the deflection at the end B, having (l, m, n) components given by $(\delta_{lB}, \delta_{mB}, 0)$ may be referred to the (x, y, z) axes by use of equations (7) and (8).

Thus

$$\begin{bmatrix} \delta_{l_B} \\ \delta_{m_B} \\ 0 \end{bmatrix} = \begin{bmatrix} \mathbf{M} \end{bmatrix} \quad \begin{bmatrix} \delta_{x_B} \\ \delta_{y_B} \\ \delta_{z_B} \end{bmatrix} \quad \cdot \cdot \cdot \quad (13)$$

and

$$\begin{bmatrix} \delta_{x_B} \\ \delta_{y_B} \\ \delta_{z_B} \end{bmatrix} = \begin{bmatrix} \mathbf{M} \end{bmatrix}^{-1} \begin{bmatrix} \delta_{l_B} \\ \delta_{m_B} \\ 0 \end{bmatrix} \quad \cdot \cdot \cdot \quad (14)$$

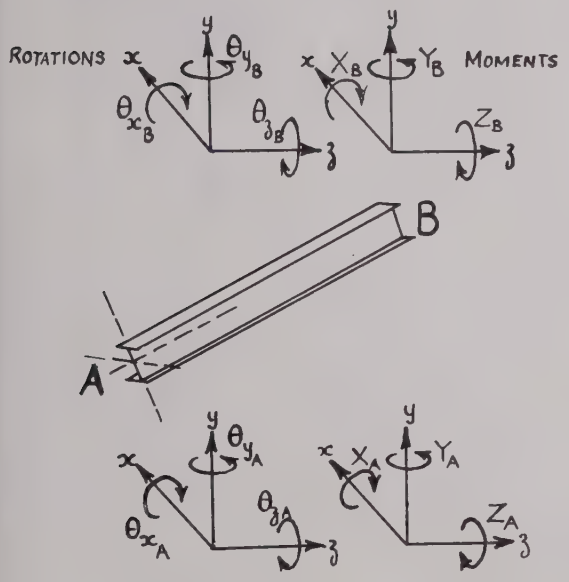

Fig. 3

It is convenient at this stage to relate the changes in moments X_A, Y_A and Z_A, applied to the end A of the member about the (x, y, z) axes to the corresponding changes in the slopes θ_{x_A}, θ_{y_A} and θ_{z_A}, produced about these axes, as shown in Fig. 3. In order to achieve this simply it is best to form the relationship between moment changes and rotations when both are referred to the (l, m, n) set and to transform both moments and rotations into (x, y, z) components by equations (10) and (11). Thus, from equations (1), (3) and (5) the (L, M, N) components of moment at A induced by unit rotations ($\varnothing_{l_A}$, $\varnothing_{m_A}$, $\varnothing_{n_A}$) are given by the terms of the matrix

$$\begin{bmatrix} 4B_l & 0 & 0 \\ 0 & 4B_m & 0 \\ 0 & 0 & B_n \end{bmatrix}$$

Resolving both moments and rotations into the (x, y, z) set, it will be seen that the (X, Y, Z) components of moment at A developed by unit rotations (θ_{x_A}, θ_{y_A}, θ_{z_A}) are given by the product

$$\begin{bmatrix} \mathbf{M} \end{bmatrix}^{-1} \begin{bmatrix} 4B_l & 0 & 0 \\ 0 & 4B_m & 0 \\ 0 & 0 & B_n \end{bmatrix} \begin{bmatrix} \mathbf{M} \end{bmatrix} \cdot \quad (15)$$

This expression if multiplied out, results in a 3×3 matrix of stiffness factors for the member AB, and will subsequently be referred to as the "Stiffness Matrix" for AB, and denoted by $\mathbf{S}_{AB}$. It replaces the single flexural rigidity term applying for each end of each member in the normal plane moment distribution procedure. As only the case of prismatic members is being treated, the matrix will be applicable to either end.

In addition it is necessary to examine the (X, Y, Z) components of moments induced at the other end B of the member by the unit rotation components (θ_{x_A}, θ_{y_A}, θ_{z_A}). By similar reasoning to that outlined above, using the results of equations (2), (4), (6), (10) and (11), it is possible to write these as the terms of the product

$$\begin{bmatrix} \mathbf{M} \end{bmatrix}^{-1} \begin{bmatrix} 2B_l & 0 & 0 \\ 0 & 2B_m & 0 \\ 0 & 0 & -B_n \end{bmatrix} \begin{bmatrix} \mathbf{M} \end{bmatrix} \cdot \cdot \quad (16)$$

This matrix will be referred to as the "Induction Matrix" for the member AB, and denoted by $\mathbf{I}_{AB}$. It is of interest to note that both the stiffness and induction matrices are symmetrical about the leading diagonal, a consequence of the Reciprocal Theorem.

III. Application of Analysis to Space Frames

The foregoing results may be applied to the solution of rigid space frames with little modification. Consider a number of members AB, AC, AD, etc., rigidly connected at a typical joint A. Each member will have a stiffness matrix and an induction matrix which will, at this stage, have been evaluated. If the joint A is clamped and an unbalanced system of moments with components (X_A, Y_A, and Z_A) is acting on the joint from the members, this system may be neutralised by rotating the joint through component angles (θ_x, θ_y and θ_z) where

$$\begin{bmatrix} \mathbf{S}_{AB} \end{bmatrix} \begin{bmatrix} \theta_x \\ \theta_y \\ \theta_z \end{bmatrix} + \begin{bmatrix} \mathbf{S}_{AC} \end{bmatrix} \begin{bmatrix} \theta_x \\ \theta_y \\ \theta_z \end{bmatrix} + + + + \begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix} = 0$$

$$\cdot \quad \cdot \quad \cdot \quad \cdot \quad (17)$$

If the corresponding terms of the various stiffness matrices are added and the sign of every term is then reversed to give a new matrix $[-\Sigma\mathbf{S}_A]$, referred to as the "Distribution Matrix" for joint A, this equation may now be rewritten

$$\begin{bmatrix} -\Sigma\mathbf{S}_A \end{bmatrix} \begin{bmatrix} \theta_x \\ \theta_y \\ \theta_z \end{bmatrix} = \begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix} \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (18)$$

which may be inverted to give

$$\begin{bmatrix} \theta_x \\ \theta_y \\ \theta_z \end{bmatrix} = \begin{bmatrix} -\Sigma\mathbf{S}_A \end{bmatrix}^{-1} \begin{bmatrix} X_A \\ Y_A \\ Z_A \end{bmatrix} \quad \cdot \quad \cdot \quad \cdot \quad (19)$$

where $[-\Sigma\mathbf{S}_A]^{-1}$ is the reciprocal matrix of $[-\Sigma\mathbf{S}_A]$.

It is best to commence the distribution as in the conventional two-dimensional case, by carrying out a first analysis allowing all joints to rotate but not to translate, the various external loadings being applied to the frame. Thus initial fixed-ended moments for each member are found and resolved, by means of equation (10) into their (x, y, z) components, and then entered at the top of the distribution table. The first joint relaxation cycle may now be completed, and the necessary rotation components (θ_x, θ_y and θ_z) found, using equation (18), to give equilibrium of each joint. These components may then be entered in the table and the induced moments at the other ends of members found, by means of the induction matrix in equation (16). This, of course, is equivalent to the "carry-over" process well known in the simple type of analysis. At this stage the various components of unbalanced moment at each joint are each summed to give the resultant unbalanced moment components, and a new cycle of joint relaxation and carry-over may be completed. After several such cycles the distribution will have converged sufficiently, and the total rotation components at each joint may be found if required,

from summation of the incremental rotations at each cycle. Finally, the resultant components of moment are found for each member, and these are resolved into components about the axes of the member. They may alternatively be found by considering the total components of rotation at each joint in association with the various stiffness matrices. It should, however, be stressed that as the distribution process considers changes in the moments from the initial conditions, the initial fixed-ended moments must always be algebraically added to the results obtained from the distribution process.

In cases where joints in the structure are free to translate it is necessary, in addition, to perform distributions analogous to the " sway distributions " encountered in plane moment distribution. In each of these, the frame must first be displaced in such a way that the distances between nodes remains unaltered and all joints are orientated parallel to their undeflected state. The moments thus induced at the ends of each member are then distributed in the manner similar to that outlined above. If the structure has many degrees of freedom there will be a number of independent sway distributions, and the results of these have to be added to the moments found from the first distribution in such proportions that the whole structure, and each part of it, is in equilibrium. The solution of the simultaneous equations involved in this process is best carried out by an iterative process if the number of independent sways becomes large.

IV. The Determination of Secondary Moments in Rigid Triangulated Space Frames

The moment distribution procedure, as outlined above, is also useful for investigating the secondary moments arising from the deformation of axially loaded triangulated space frames. The axial loads must first be found by Southwell's method of ' tension coefficients ' or some other convenient procedure, assuming the frame to be pin-jointed, and the consequent displacements of the nodes determined. This may be done in a number of ways ; strain energy is used in the example quoted later in the paper. Having determined all displacements it is now possible to refer these, for each member, to its relevant (l, m, n) axes, and thus to determine the quantities δ_l and δ_m, defined in § II for the typical member AB. The initial moments induced in the members are given in equations (1) to (4). Thus

$$\begin{bmatrix} L_A \\ M_A \\ N_A \end{bmatrix} = \begin{bmatrix} L_B \\ M_B \\ N_B \end{bmatrix} = \frac{6E}{S^2} \begin{bmatrix} +I_l\,\delta_{mB} \\ -I_m\,\delta_{lB} \\ 0 \end{bmatrix} \quad (20)$$

It is now a simple matter to resolve these moment components into the (x, y, z) directions, by equation (10) and to distribute them in the usual way. The shears associated with the final moments thus obtained may cause a slight unbalance in the axial force distribution and in extreme cases it may be necessary to re-analyse the frame using successive approximations. This will not be necessary, however, when the great proportion of the load is carried axially, and the results obtained by one application of the above procedure should be sufficiently accurate for most practical purposes.

V. Application of the Theory to Typical Examples

(a). It is useful to illustrate the method firstly with a simple example in order to demonstrate the basic

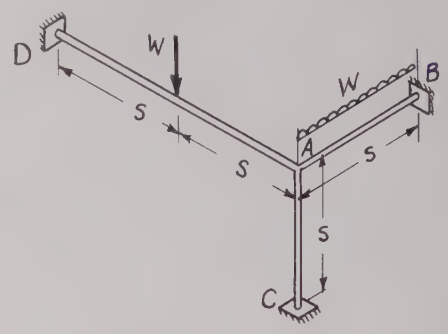

Fig. 4

distribution and carry-over procedure. For this purpose a frame has been selected, consisting of three tubular steel members of identical cross-section, having the dimensions and loading shown in Fig. 4. The joint A is assumed rigid, all ends are rigidly built-in and the three members are mutually perpendicular.

As the members of the frame are tubular, the problem is at once simplified by the infinite choice available in the selection of cross-sectional bending axes. Thus, for convenience these have been taken as shown in Fig. 5, which also shows the reference set (x, y, z).

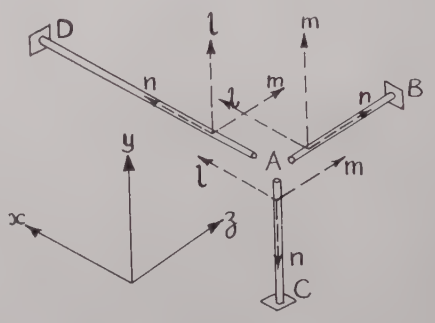

Fig. 5

The matrices of direction cosines for each member may now be written down as follows :

$$\mathbf{M}_{AB} \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix} \quad \mathbf{M}_{AC} \begin{bmatrix} 1 & 0 & 0 \\ 0 & 0 & 1 \\ 0 & -1 & 0 \end{bmatrix} \quad \mathbf{M}_{AD} \begin{bmatrix} 0 & 1 & 0 \\ 0 & 0 & 1 \\ 1 & 0 & 0 \end{bmatrix}$$

For steel, $G = 0{\cdot}4E$ and for tubular members, $I_l = I_m = 0{\cdot}5I_{(polar)}$. Hence $EI_l = EI_m = EI$, and $C = GI_{(polar)} = 0{\cdot}8EI$.

The stiffness matrices for the three members may now be written down as follows by substitution into equation (15). The common factor $\frac{EI}{S}$ (or B) has been taken out of each term and shown outside the matrix.

$$\mathbf{S}_{AB}\ B\begin{bmatrix} 4 & 0 & 0 \\ 0 & 4 & 0 \\ 0 & 0 & 0{\cdot}8 \end{bmatrix} \quad \mathbf{S}_{AC}\ B\begin{bmatrix} 4 & 0 & 0 \\ 0 & 0{\cdot}8 & 0 \\ 0 & 0 & 4 \end{bmatrix} \quad \mathbf{S}_{AD}\ B\begin{bmatrix} 0{\cdot}4 & 0 & 0 \\ 0 & 2 & 0 \\ 0 & 0 & 2 \end{bmatrix}$$

DISTRIBUTION OF MOMENTS IN SIMPLE FRAME

	Joint A			Joint B	Joint C	Joint D	
	Member AB	Member AC	Member AD	Member AB	Member AC	Member AD	
	$B\begin{bmatrix}2&0&0\\0&2&0\\0&0&-8\end{bmatrix}$	$B\begin{bmatrix}2&0&0\\0&-8&0\\0&0&2\end{bmatrix}$	$B\begin{bmatrix}-4&0&0\\0&1&0\\0&0&1\end{bmatrix}$				Induction Matrices
	$B\begin{bmatrix}4&0&0\\0&4&0\\0&0&8\end{bmatrix}$	$B\begin{bmatrix}4&0&0\\0&8&0\\0&0&4\end{bmatrix}$	$B\begin{bmatrix}4&0&0\\0&2&0\\0&0&2\end{bmatrix}$				Stiffness Matrices
	$\frac{1}{B}\begin{bmatrix}-0{\cdot}1190&0&0\\0&-0{\cdot}1470&0\\0&0&-0{\cdot}1470\end{bmatrix}$						Reciprocal of Distribution Matrix
1	$\begin{bmatrix}-100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\+300\end{bmatrix}$	$\begin{bmatrix}+100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\-300\end{bmatrix}$	[X Y Z] Components of Fixed-Ended Moments
2	$\begin{bmatrix}-100\\0\\+300\end{bmatrix}$						Resultant Unbalanced Moment
3	$\frac{1}{B}\begin{pmatrix}+11{\cdot}90\\0\\-44{\cdot}10\end{pmatrix}$						Release Joint A
4				$\begin{bmatrix}+23{\cdot}80\\0\\+35{\cdot}28\end{bmatrix}$	$\begin{bmatrix}+23{\cdot}80\\0\\-88{\cdot}20\end{bmatrix}$	$\begin{bmatrix}-4{\cdot}76\\0\\-44{\cdot}10\end{bmatrix}$	Carry-Over to B, C, D.
5	$\begin{bmatrix}+47{\cdot}60\\0\\-35{\cdot}28\end{bmatrix}$	$\begin{bmatrix}+47{\cdot}60\\0\\-176{\cdot}40\end{bmatrix}$	$\begin{bmatrix}+4{\cdot}76\\0\\-88{\cdot}20\end{bmatrix}$				Moments developed at A
6	$\begin{bmatrix}-52{\cdot}40\\0\\-35{\cdot}28\end{bmatrix}$	$\begin{bmatrix}+47{\cdot}60\\0\\-176{\cdot}40\end{bmatrix}$	$\begin{bmatrix}+4{\cdot}76\\0\\+211{\cdot}80\end{bmatrix}$	$\begin{bmatrix}+123{\cdot}80\\0\\+35{\cdot}28\end{bmatrix}$	$\begin{bmatrix}+23{\cdot}80\\0\\-88{\cdot}20\end{bmatrix}$	$\begin{bmatrix}-4{\cdot}76\\0\\-344{\cdot}10\end{bmatrix}$	[X Y Z] Components of Resultant Moment
7	$\begin{bmatrix}-52{\cdot}40\\0\\-35{\cdot}28\end{bmatrix}$	$\begin{bmatrix}+47{\cdot}60\\-176{\cdot}40\\0\end{bmatrix}$	$\begin{bmatrix}0\\+211{\cdot}80\\+4{\cdot}76\end{bmatrix}$	$\begin{bmatrix}+123{\cdot}80\\0\\+35{\cdot}28\end{bmatrix}$	$\begin{bmatrix}+23{\cdot}80\\-88{\cdot}20\\0\end{bmatrix}$	$\begin{bmatrix}0\\-344{\cdot}10\\-4{\cdot}76\end{bmatrix}$	[L M N] Components of Resultant Moment

Table I

Similarly, the induction matrices for the three members are found, from equation (16), to be

$$\mathbf{I}_{AB} \qquad \mathbf{I}_{AC} \qquad \mathbf{I}_{AD}$$

$$B\begin{bmatrix}2&0&0\\0&2&0\\0&0&-0{\cdot}8\end{bmatrix} \quad B\begin{bmatrix}2&0&0\\0&-0{\cdot}8&0\\0&0&2\end{bmatrix} \quad B\begin{bmatrix}-0{\cdot}4&0&0\\0&1&0\\0&0&1\end{bmatrix}$$

The distribution matrix for joint A, $[-\Sigma\mathbf{S}_A]$, found by adding $\mathbf{S}_{AB}$, $\mathbf{S}_{AC}$ and $\mathbf{S}_{AD}$ and changing the signs of all terms is,

$$B\begin{bmatrix}-8{\cdot}4&0&0\\0&-6{\cdot}8&0\\0&0&-6{\cdot}8\end{bmatrix}$$

and its reciprocal, $[-\Sigma\mathbf{S}_A]^{-1}$ is given* by

$$\frac{1}{B}\begin{bmatrix}-0{\cdot}1190&0&0\\0&-0{\cdot}1470&0\\0&0&-0{\cdot}1470\end{bmatrix}$$

It will be appreciated that all of these matrices depend upon the properties of the structure, and not upon the loading to which it is subjected.

*The procedure for determining the reciprocal of a matrix depends to a large extent on the complexity of the matrix itself and on the degree of accuracy required.

One approach to the problem is to determine the co-factor of every element in the matrix **M** and to enter this as the corresponding element in a new matrix which is referred to as the "adjoint" of **M**. The "adjugate" must now be found by transposing rows and columns in the adjoint. In the case of symmetrical matrices the adjoint and adjugate will be equal. The reciprocal matrix is then the matrix of terms found by dividing each term in the adjugate by the determinant of the matrix **M**. For a fuller treatment of this the reader is referred to Reference (7).

The fixed-ended moments developed in the members AB and AD may now be determined. Considering first the moments in AB,

$$L_{(F)B} = -L_{(F)A} = \frac{WS}{12}$$

and, for AD,

$$M_{(F)A} = -M_{(F)D} = \frac{2WS}{8}$$

For convenience in computation, these are given the numerical values of 100 units and 300 units respectively. It is now possible to distribute the moments by relaxing joint A and carrying-over the moments to B, C and D. This procedure is shown in Table I, which has purposely been spread out in order to show the process clearly. For exposition, moments have been enclosed in brackets [ ] and joint rotations in parentheses ( ). The (X, Y, Z) components of fixed-ended moment are entered in row (1), the resultant unbalanced moment at A in row (2) being merely the sum of the three terms for the separate members. The joint A is now released and the components of rotation, found by computing the product of the reciprocal matrix with the column matrix in row (2), are shown in row (3). The moments carried over to the built-in ends B, C and D are next determined, using the appropriate induction matrices, and these are entered in row (4). The moment change induced in each member at A may also be

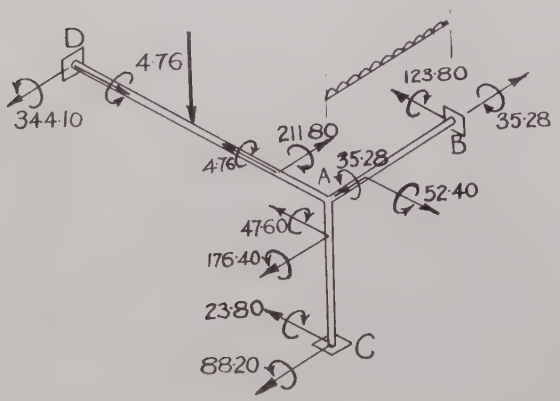

Fig. 6

found as in row (5), using the stiffness matrices. As there is only one joint to relax in this frame, the distribution process is now complete and it is merely necessary to sum the moments to obtain the resultants shown in row (6). These may be referred to the (l, m, n) axes of each member by use of equation (9), so that the final moments, expressed as percentages of $\frac{WS}{12}$ are derived as in row (7). They are also shown, for convenience, on a sketch of the frame, in Fig. 6. The results obtained will be found to be in excellent agreement with those derived by an independent elastic analysis. It is of interest to observe that for this simple orthogonal frame, there is no interaction between moments in different planes, and the method reduces to the simultaneous performance of two plane distributions.

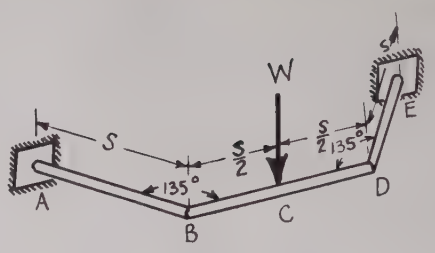

Fig. 7

(*b*) As a second example, the symmetrical bow-girder $ABCDE$, shown in Fig. 7 has been selected. This consists of three identical tubular steel members, each of length S, and is such that the ends A and E are rigidly built-in, and the joints at B and D are rigid. Proceeding as in the first example, the member axes have been selected to be horizontal and vertical, and the (x, y, z) axis set has been chosen as shown in Fig. 8.

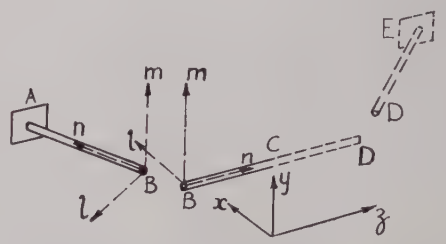

Fig. 8

It is wise to take full advantage of the symmetrical conditions in this problem, thereby saving a considerable amount of work. Attention will therefore be confined to the portion ABC. The matrices of direction cosines for these members are

$$\mathbf{M}_{AB} = \begin{bmatrix} -\frac{1}{\sqrt{2}} & 0 & -\frac{1}{\sqrt{2}} \\ 0 & 1 & 0 \\ +\frac{1}{\sqrt{2}} & 0 & -\frac{1}{\sqrt{2}} \end{bmatrix} \qquad \mathbf{M}_{BD} = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix}$$

Using the same relationships $C = 0{\cdot}8EI$ as in the first example, the stiffness matrices for members AB and BD may be evaluated at this stage, to give

$$\mathbf{S}_{AB} = B\begin{bmatrix} 2{\cdot}4 & 0 & 1{\cdot}6 \\ 0 & 4 & 0 \\ 1{\cdot}6 & 0 & 2{\cdot}4 \end{bmatrix} \qquad \mathbf{S}_{BD} = B\begin{bmatrix} 4 & 0 & 0 \\ 0 & 4 & 0 \\ 0 & 0 & 0{\cdot}8 \end{bmatrix}$$

In the case of the member BD the stiffness matrix must be modified if it is intended to dispense with the carry-over procedure to the right hand side of the frame. It will be appreciated that the major and minor axis stiffnesses must be reduced by a factor of 2 in order to achieve this, as this modification will then give the same moments as those developed by

one joint relaxation less than 50 per cent carry-over from the opposite end. The torsional stiffness must be regarded as zero for this member, however, as no torsion can develop in BD at any stage under the symmetrical loading condition. (These modifications may be compared with the well known case in plane moment distribution in which the $\frac{3}{4}$ term is sometimes introduced to avoid the necessity to balance and carry-over from a pin end). The modified stiffness matrix for the central member is thus,

$$\mathbf{S}_{BC}$$
$$B\begin{bmatrix} 2 & 0 & 0 \\ 0 & 2 & 0 \\ 0 & 0 & 0 \end{bmatrix}$$

The induction matrix for member AB may be found from equation (16) to be

$$\mathbf{I}_{AB}$$
$$B\begin{bmatrix} 0{\cdot}6 & 0 & 1{\cdot}4 \\ 0 & 2 & 0 \\ 1{\cdot}4 & 0 & 0{\cdot}6 \end{bmatrix}$$

Since there is to be no carrying-over in BD, the induction matrix loses its significance for this member.

The distribution matrix for joint B becomes

$$[-\Sigma\mathbf{S}_B] = B\begin{bmatrix} -4{\cdot}4 & 0 & -1{\cdot}6 \\ 0 & -6 & 0 \\ -1{\cdot}6 & 0 & -2{\cdot}4 \end{bmatrix}$$

and its reciprocal is

$$[-\Sigma\mathbf{S}_B]^{-1} = \frac{1}{B}\begin{bmatrix} -0{\cdot}30 & 0 & +0{\cdot}20 \\ 0 & -0{\cdot}1667 & 0 \\ +0{\cdot}20 & 0 & -0{\cdot}55 \end{bmatrix}$$

Consider now the moments developed in the bow girder when the central vertical load W is applied at C, and the joints B and D are supported in order that they do not deflect downwards. The fixed-ended moment at B is, of course, $\frac{-WS}{8}$, with a horizontal axis and has been given the numerical value of 100 units, as shown in the first distribution (Table II). The distribution process is also simple for this problem; there is only one joint to be released, and one carry-over to the built-in support, at which stage the final moments may be determined. These have been resolved back to the (l, m, n) axis set for each member, the results being entered in the last line of Table II and shown in Fig. 9.

Bow Girder - First Distribution

Joint A	Joint B		
	Member AB	Member BD	
	$B\begin{bmatrix} 0{\cdot}6 & 0 & 1{\cdot}4 \\ 0 & 2 & 0 \\ 1{\cdot}4 & 0 & 0{\cdot}6 \end{bmatrix}$		Induction Matrix
	$B\begin{bmatrix} 2{\cdot}4 & 0 & 1{\cdot}6 \\ 0 & 4 & 0 \\ 1{\cdot}6 & 0 & 2{\cdot}4 \end{bmatrix}$	$B\begin{bmatrix} 2 & 0 & 0 \\ 0 & 2 & 0 \\ 0 & 0 & 0 \end{bmatrix}$	Modified Stiffness Matrices
	$\frac{1}{B}\begin{bmatrix} -0{\cdot}30 & 0 & +0{\cdot}20 \\ 0 & -0{\cdot}1667 & 0 \\ +0{\cdot}20 & 0 & -0{\cdot}55 \end{bmatrix}$		Reciprocal of Distribution Matrix
$\begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$	$\begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$	$\begin{bmatrix} -100 \\ 0 \\ 0 \end{bmatrix}$	Initial Moments
	$\frac{1}{B}\begin{pmatrix} +30 \\ 0 \\ -20 \end{pmatrix}$		Release Joint B
$\begin{bmatrix} -10 \\ 0 \\ +30 \end{bmatrix}$			Carry-Over to A
	$\begin{bmatrix} +40 \\ 0 \\ 0 \end{bmatrix}$	$\begin{bmatrix} +60 \\ 0 \\ 0 \end{bmatrix}$	Moments Developed at B
$\begin{bmatrix} -10 \\ 0 \\ +30 \end{bmatrix}$	$\begin{bmatrix} +40 \\ 0 \\ 0 \end{bmatrix}$	$\begin{bmatrix} -40 \\ 0 \\ 0 \end{bmatrix}$	X Components Y of Resultant Z Moment
$\begin{bmatrix} -14{\cdot}14 \\ 0 \\ -28{\cdot}28 \end{bmatrix}$	$\begin{bmatrix} -28{\cdot}28 \\ 0 \\ +28{\cdot}28 \end{bmatrix}$	$\begin{bmatrix} -40 \\ 0 \\ 0 \end{bmatrix}$	L Components M of Resultant N Moment

Table II

In order to compute the additional moments caused by the removal of the supports at B and D, equal downward deflections of arbitrary magnitude have been applied to these joints. This results in the initial moments in the member AB (for the half-frame being analysed) shown entered at the top of the sway distribution (Table III). These have been distributed in the same way as was done in the first distribution; and the results are shown in Fig. 10, after having

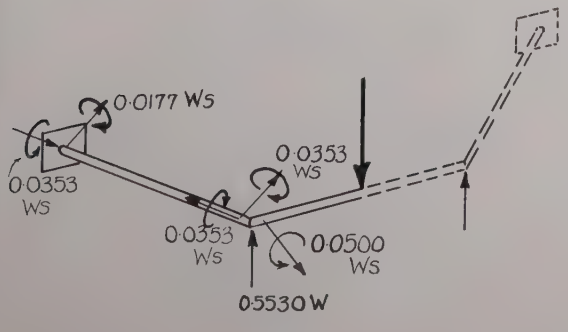

Fig. 9

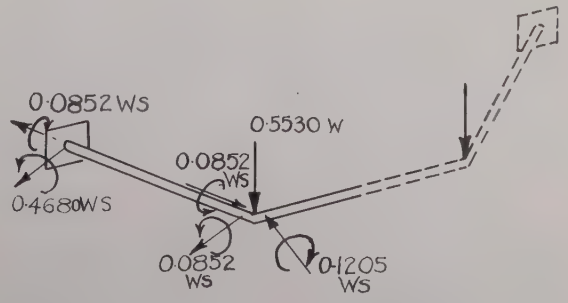

Fig. 10

BOW GIRDER – SECOND DISTRIBUTION

JOINT A	JOINT B		
MEMBER AB		MEMBER BD	
$\begin{bmatrix}-100\\0\\-100\end{bmatrix}$	$\begin{bmatrix}-100\\0\\-100\end{bmatrix}$	$\begin{bmatrix}0\\0\\0\end{bmatrix}$	Initial Moments
	$\frac{1}{B}\begin{pmatrix}+10\\0\\+35\end{pmatrix}$		Release Joint B
$\begin{bmatrix}+55\\0\\+35\end{bmatrix}$			Carry-Over to A
	$\begin{bmatrix}+80\\0\\+100\end{bmatrix}$	$\begin{bmatrix}+20\\0\\0\end{bmatrix}$	Moments Developed at B
$\begin{bmatrix}-45\\0\\-65\end{bmatrix}$	$\begin{bmatrix}-20\\0\\0\end{bmatrix}$	$\begin{bmatrix}+20\\0\\0\end{bmatrix}$	[X] Components [Y] of Resultant [Z] Moment
$\begin{bmatrix}+77{\cdot}79\\0\\+14{\cdot}14\end{bmatrix}$	$\begin{bmatrix}+14{\cdot}14\\0\\-14{\cdot}14\end{bmatrix}$	$\begin{bmatrix}+20\\0\\0\end{bmatrix}$	[L] Components [M] of Resultant [N] Moment

Table III

been scaled up to the correct extent to neutralise the effect of the reactions at B and D. The final system of moments has been obtained by combining the results of the two distributions in such a way that the bow-girder is in equilibrium without external reactions at B and D, and is shown in Fig. 11.

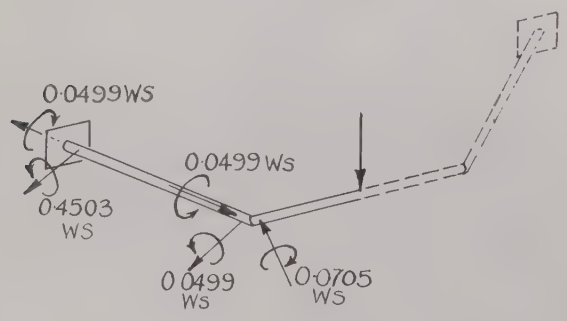

Fig. 11

(c) For the third example, the secondary flexural stresses arising from the deformation of an axially loaded tripod are examined. As shown in Fig. 12, the rigid steel tripod, $OABC$ is rigidly connected at O and is rigidly fixed at the base supports A, B and C which are in the same horizontal plane. The joint O is subjected to two horizontal loads, P and $\frac{P}{2}$, at right angles. All members are assumed to be of identical dimensions and of tubular section. For the analysis the axes for each member and for the distribution have been selected as shown in Fig. 13. The direction cosine matrices for the members thus become

$$\mathbf{M}_{OA}=\begin{bmatrix}\frac{+\sqrt{2}}{\sqrt{3}} & 0 & \frac{+1}{\sqrt{3}}\\ 0 & -1 & 0\\ \frac{+1}{\sqrt{3}} & 0 & \frac{-\sqrt{2}}{\sqrt{3}}\end{bmatrix}\qquad \mathbf{M}_{OB}=\begin{bmatrix}\frac{-1}{\sqrt{6}} & \frac{-1}{\sqrt{2}} & \frac{+1}{\sqrt{3}}\\ \frac{-\sqrt{3}}{2} & \frac{+1}{2} & 0\\ \frac{-1}{2\sqrt{3}} & \frac{-1}{2} & \frac{-\sqrt{2}}{\sqrt{3}}\end{bmatrix}$$

$$\mathbf{M}_{OC}=\begin{bmatrix}\frac{-1}{\sqrt{6}} & \frac{+1}{\sqrt{2}} & \frac{+1}{\sqrt{3}}\\ \frac{+\sqrt{3}}{2} & \frac{+1}{2} & 0\\ \frac{-1}{2\sqrt{3}} & \frac{+1}{2} & \frac{-\sqrt{2}}{\sqrt{3}}\end{bmatrix}$$

The stiffness and induction matrices evaluate as :

$$\mathbf{S}_{OA}\quad \frac{EI}{S}\begin{bmatrix}2{\cdot}933 & 0 & 1{\cdot}511\\ 0 & 4 & 0\\ 1{\cdot}511 & 0 & 1{\cdot}866\end{bmatrix}\qquad \mathbf{S}_{OB}\quad \frac{EI}{S}\begin{bmatrix}3{\cdot}733 & -0{\cdot}462 & -0{\cdot}755\\ -0{\cdot}462 & +3{\cdot}200 & -1{\cdot}307\\ -0{\cdot}755 & -1{\cdot}307 & +1{\cdot}867\end{bmatrix}$$

$$\mathbf{S}_{OC}\quad \frac{EI}{S}\begin{bmatrix}3{\cdot}733 & +0{\cdot}462 & -0{\cdot}755\\ +0{\cdot}462 & +3{\cdot}200 & +1{\cdot}307\\ -0{\cdot}755 & +1{\cdot}307 & +1{\cdot}867\end{bmatrix}$$

$$\mathbf{I}_{OA}\quad \frac{EI}{S}\begin{bmatrix}+1{\cdot}067 & 0 & +1{\cdot}320\\ 0 & 2 & 0\\ +1{\cdot}320 & 0 & +0{\cdot}133\end{bmatrix}\qquad \mathbf{I}_{OB}\quad \frac{EI}{S}\begin{bmatrix}+1{\cdot}767 & -0{\cdot}404 & -0{\cdot}660\\ -0{\cdot}404 & +1{\cdot}300 & -1{\cdot}143\\ -0{\cdot}660 & -1{\cdot}143 & +0{\cdot}133\end{bmatrix}$$

$$\mathbf{I}_{OC}\quad \frac{EI}{S}\begin{bmatrix}+1{\cdot}767 & +0{\cdot}404 & -0{\cdot}660\\ +0{\cdot}404 & +1{\cdot}300 & +1{\cdot}143\\ -0{\cdot}660 & +1{\cdot}143 & +0{\cdot}133\end{bmatrix}$$

Hence the distribution matrix at O becomes

$$\frac{EI}{S}\begin{bmatrix}-10{\cdot}400 & 0 & 0\\ 0 & -10{\cdot}400 & 0\\ 0 & 0 & -5{\cdot}600\end{bmatrix}$$

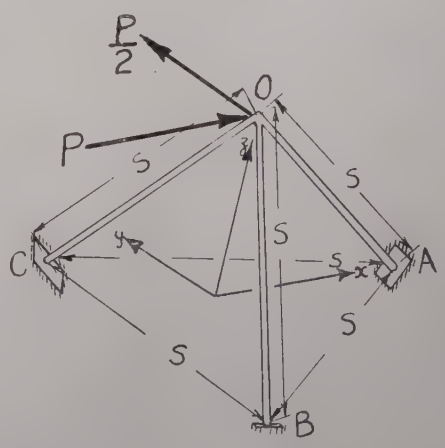

Fig. 12

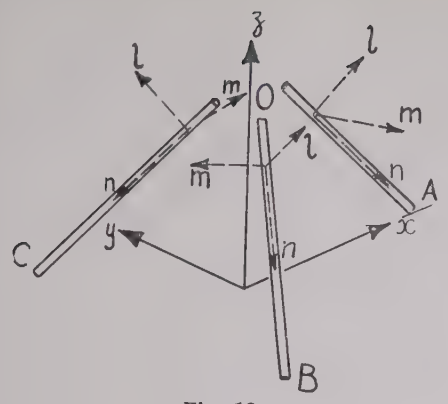

Fig. 13

and its reciprocal is the following diagonal matrix;

$$\frac{S}{EI}\begin{bmatrix} -0\cdot0961 & 0 & 0 \\ 0 & -0\cdot0961 & 0 \\ 0 & 0 & -0\cdot1782 \end{bmatrix}$$

Having determined the induction and distribution matrices it is now necessary to determine the initial fixed-ended moments developed by the displacement of the apex joint O. Loads P, Q and R have, for this purpose been applied at O in the (x, y, z) directions, in order that the three displacement components might be found by strain energy methods.

As pointed out in § IV it is first necessary to determine the axial loads, and an interesting variation of the tension coefficient method is to resolve by means of the direction cosine matrices. Thus the compressive forces N_{OA}, N_{OB} and N_{OC} in the corresponding members may be derived from the relation,

$$\begin{bmatrix} P \\ Q \\ R \end{bmatrix} = \left[\mathbf{M}\right]^{-1}_{OA}\begin{bmatrix} 0 \\ 0 \\ N_{OA} \end{bmatrix} + \left[\mathbf{M}\right]^{-1}_{OB}\begin{bmatrix} 0 \\ 0 \\ N_{OB} \end{bmatrix} + \left[\mathbf{M}\right]^{-1}_{OC}\begin{bmatrix} 0 \\ 0 \\ N_{OC} \end{bmatrix}$$

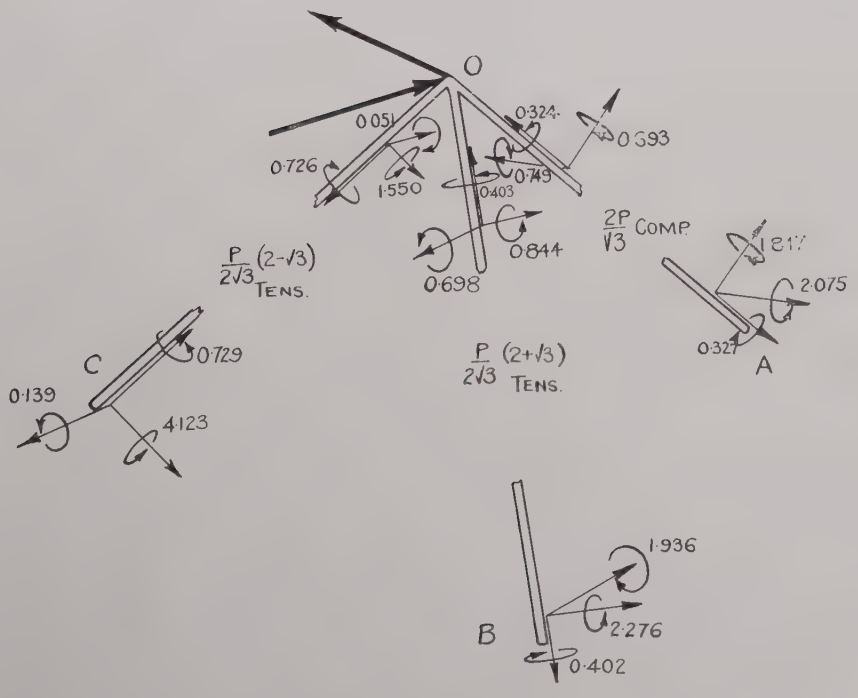

Fig. 14

Distribution of Moments in Tripod

OA: $\frac{EI}{S}\begin{bmatrix} 1{\cdot}067 & 0 & 1{\cdot}320 \\ 0 & 2 & 0 \\ 1{\cdot}320 & 0 & 0{\cdot}133 \end{bmatrix}$ OB: $\frac{EI}{S}\begin{bmatrix} 1{\cdot}767 & -0{\cdot}404 & -0{\cdot}660 \\ -0{\cdot}404 & +1{\cdot}300 & -1{\cdot}143 \\ -0{\cdot}660 & -1{\cdot}143 & +0{\cdot}133 \end{bmatrix}$ OC: $\frac{EI}{S}\begin{bmatrix} 1{\cdot}767 & +0{\cdot}404 & -0{\cdot}660 \\ 0{\cdot}404 & +1{\cdot}300 & +1{\cdot}143 \\ -0{\cdot}660 & +1{\cdot}143 & +0{\cdot}133 \end{bmatrix}$ — Induction Matrices

$\frac{EI}{S}\begin{bmatrix} 2{\cdot}933 & 0 & 1{\cdot}511 \\ 0 & 4 & 0 \\ 1{\cdot}511 & 0 & 1{\cdot}866 \end{bmatrix}$ $\frac{EI}{S}\begin{bmatrix} 3{\cdot}733 & -0{\cdot}462 & -0{\cdot}755 \\ -0{\cdot}462 & +3{\cdot}200 & +1{\cdot}307 \\ -0{\cdot}755 & -1{\cdot}307 & +1{\cdot}867 \end{bmatrix}$ $\frac{EI}{S}\begin{bmatrix} 3{\cdot}733 & +0{\cdot}462 & -0{\cdot}755 \\ +0{\cdot}462 & +3{\cdot}200 & +1{\cdot}307 \\ -0{\cdot}755 & +1{\cdot}307 & +1{\cdot}867 \end{bmatrix}$ — Stiffness Matrices

$\frac{S}{EI}\begin{bmatrix} -0{\cdot}0961 & 0 & 0 \\ 0 & -0{\cdot}0961 & 0 \\ 0 & 0 & -0{\cdot}1782 \end{bmatrix}$ — Reciprocal of Distribution Matrix for O

Joint A	Joint B	Joint C	Joint O		
Member OA	Member OB	Member OC	Member OA	Member OB	Member OC
$+\sqrt{6}$	$+\sqrt{6}$	$+\sqrt{6}$	$+\sqrt{6}$	$+\sqrt{6}$	$+\sqrt{6}$
$-2\sqrt{6}$	$-2\sqrt{6}$	$-2\sqrt{6}$	$-2\sqrt{6}$	$-2\sqrt{6}$	$-2\sqrt{6}$
$+\sqrt{3}$	$-\frac{\sqrt{3}}{2}(1-2\sqrt{3})$	$-\frac{\sqrt{3}}{2}(1+2\sqrt{3})$	$+\sqrt{3}$	$-\frac{\sqrt{3}}{2}(1-2\sqrt{3})$	$-\frac{\sqrt{3}}{2}(1+2\sqrt{3})$
+2·450	+2·450	+2·450	+2·450	+2·450	+2·450
−4·899	−4·899	−4·899	−4·899	−4·899	−4·899
+1·732	+2·134	−3·866	+1·732	+2·134	−3·666
				+7·350	
				−14·697	
				0	
			$\frac{2P}{AE}$		
			−0·706		
			+1·412		
			0		
−0·753	−1·818	−0·677			
+2·824	+2·121	+1·550			
−0·932	−1·148	+2·080			
			−2·071	−3·287	−1·983
			+5·648	+4·844	+4·192
			−1·067	−1·312	+2·378
+1·697	+0·632	+1·773	+0·379	−0·837	+0·467
−2·075	−2·778	−3·349	+0·749	−0·055	−0·707
+0·800	+0·986	−1·786	+0·665	+0·822	−1·488
+1·847	+2·276	−4·123	+0·693	+0·844	−1·550
+2·075	−1·936	−0·139	−0·749	+0·698	+0·051
+0·327	+0·402	−0·729	−0·324	−0·403	+0·726

Initial Moments (Note that all moments are to be multiplied by $\frac{2P}{AS}$)

Resultant Unbalanced Moment at O

Release Joint O

Carry-Over to A, B, C.

Moments Developed at O

X Components Y of Resultant Z Moment

L Components M of Resultant N Moment

Table IV

giving

$$N_{OA} = \frac{\sqrt{3}}{6}(4P - \sqrt{2}R)$$

$$N_{OB} = -\frac{\sqrt{3}}{6}(2P + 2\sqrt{3}Q + \sqrt{2}R)$$

$$N_{OC} = -\frac{\sqrt{3}}{6}(2P - 2\sqrt{3}Q + \sqrt{2}R)$$

$$\delta_{xo} = \frac{\partial U}{\partial P} =$$

$$\frac{S}{EA}\left\{\begin{array}{l}\left(\frac{4\sqrt{3}}{6}\right)\left(\frac{\sqrt{3}}{6}\right)\left(4P - \sqrt{2}R\right) \\ +\left(\frac{-2\sqrt{3}}{6}\right)\left(\frac{-\sqrt{3}}{6}\right)\left(2P + 2\sqrt{3}Q + \sqrt{2}R\right) \\ +\left(\frac{-2\sqrt{3}}{6}\right)\left(\frac{-\sqrt{3}}{6}\right)\left(2P - 2\sqrt{3}Q + \sqrt{2}R\right)\end{array}\right\}$$

Making use of the fact that $Q = \frac{P}{2}$, $R = 0$,

$$\delta_{xo} = \frac{+2PS}{EA}$$

Similarly $\delta_{yo} = +\frac{PS}{EA}$.

$$\delta_{zo} = 0.$$

Hence for member OA,

$$\begin{bmatrix}\delta_l \\ \delta_m \\ \delta_n\end{bmatrix} = \frac{PS}{EA}\begin{bmatrix}\frac{\sqrt{2}}{\sqrt{3}} & 0 & \frac{1}{\sqrt{3}} \\ 0 & -1 & 0 \\ \frac{1}{\sqrt{3}} & 0 & \frac{-\sqrt{2}}{\sqrt{3}}\end{bmatrix}\begin{bmatrix}2 \\ 1 \\ 0\end{bmatrix} = \frac{PS}{EA}\begin{bmatrix}\frac{2\sqrt{2}}{3} \\ -1 \\ \frac{2}{\sqrt{3}}\end{bmatrix}$$

and, from equations (20), (10)

$$\begin{bmatrix}X \\ Y \\ Z\end{bmatrix} = \begin{bmatrix}\\ \mathbf{M} \\ \\ \end{bmatrix}^{-1}\begin{bmatrix}L \\ M \\ N\end{bmatrix}$$

$$= \frac{2EI}{S^2}\cdot\frac{PS}{EA}\begin{bmatrix}\frac{\sqrt{2}}{\sqrt{3}} & 0 & \frac{1}{\sqrt{3}} \\ 0 & -1 & 0 \\ \frac{+1}{\sqrt{3}} & 0 & \frac{-\sqrt{2}}{\sqrt{3}}\end{bmatrix}\begin{bmatrix}+3 \\ +6\frac{\sqrt{2}}{\sqrt{3}} \\ 0\end{bmatrix}$$

$$= \frac{2IP}{AS}\begin{bmatrix}+3\frac{\sqrt{2}}{\sqrt{3}} \\ -6\frac{\sqrt{2}}{\sqrt{3}} \\ +\sqrt{3}\end{bmatrix}$$

In a similar way, it can be shown that, for OB,

$$\begin{bmatrix}X \\ Y \\ Z\end{bmatrix} = \frac{2IP}{AS}\begin{bmatrix}\sqrt{6} \\ -4\frac{\sqrt{3}}{\sqrt{2}} \\ -\frac{\sqrt{3}}{2}(1 - 2\sqrt{3})\end{bmatrix}$$

and for OC

$$\begin{bmatrix}X \\ Y \\ Z\end{bmatrix} = \frac{2IP}{AS}\begin{bmatrix}\sqrt{6} \\ -4\frac{\sqrt{3}}{\sqrt{2}} \\ -\frac{\sqrt{3}}{2}(1 + 2\sqrt{3})\end{bmatrix}$$

The distribution may now be undertaken, as shown in Table IV, the final moments (expressed as coefficients of $\left(\frac{2IP}{AS}\right)$) and axial loads being shown in Fig. 14.

VI Conclusions

The foregoing examples will illustrate the process of solution of space frames by moment distribution. It will be apparent from the relative simplicity of the method that it affords a convenient means of analysis. This is especially so in the case of space frames when the order of redundancy is large, and the number of degrees of freedom (and consequently the number of independent sway distributions) is small, as illustrated by the tripod problem. The majority of the computation is in the evaluation of the distribution and induction matrices, which, when once found, do not change for different loading conditions. Thus the method is quite convenient for problems involving moving loads. Even if it is desired to make some slight modifications to the structure itself, it is probable that few of the matrix terms will need to be altered, so that corrections to the matrices and their reciprocals may be made relatively easily.

Acknowledgements

The Author wishes to thank Mr. D. Campbell-Allen and Mr. P. B. Jones for their helpful suggestions and constructive criticism.

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Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper and also the following paper on pp. 196-199. Communications intended for publication should be forwarded to reach the Institution by 31st August, 1960.

Uniform Load on a Slab or Plate Stiffened by Beams

By M. Holmes, B.Sc., Ph.D., A.M.I.C.E.

Synopsis

The paper describes a method of analysis for beam and slab (or plate) systems. The theoretical analysis in its general form may be used to solve various loading conditions, but the computational work tends to be excessive. For the particular case of a uniform load over the whole area of the stiffened slab, however, the results of the analysis may be expressed as a set of coefficients the values of which may be represented graphically. For uniform loading, therefore, the method of analysis is suitable for design purposes.

Introduction

A fully referenced account has recently been published[1] of the existing types of analysis which are available for grid frameworks, beam and slab and other related structures. In this account the authors, Hendry and Jaeger, point out the disadvantages of existing applications of plate theory to beam and slab structures. Many of these theories assume that there is no shear transmission between plate and beam i.e. only vertical loads are transferred. In addition these methods involve prohibitive amounts of arithmetical computation.

The method of analysis described here makes a minimum of simplifying assumptions and, for the particular case of uniform loading, may be reduced to simple design curves. As uniform loading is present as the structure dead load, and is not uncommon as a design live load, it is worth while to develop the analysis for this particular case of loading. The theory has already been described in general terms[2], therefore,only a brief description is given here, after which the general theory is developed for the special case of uniform load.

NOTATION

l — Span.
a — Spacing of stiffening beams.
h — Thickness of plate or slab.
d — Depth of stiffening beam.
$\frac{\alpha d}{2}$ — Depth to centroid of stiffening beam.
A — Cross-sectional area of stiffening beam.
$E_B I$ — Flexural rigidity of stiffening beam.
D — Flexural rigidity per unit width of plate or slab $= \frac{E_S h^3}{12(1-\upsilon^2)}$.
υ — Poisson's Ratio for plate or slab.
m — Ratio of moduli of elasticity of beam and slab $= \frac{E_B}{E_S}$.
M,N — Bending moment and axial force acting at centroid of beam.
$f_x f_y$ — Direct stresses in the plate or slab.
$M_x M_y$ — Bending moments per unit width in the plate or slab.
q — Applied load per unit area.
w — Deflection of plate or slab.

Basis of Analysis

The beam and slab system, Fig. 1(a), is considered to be semi-infinite i.e. to consist of a large number of bays of width a. The results of the analysis thus strictly only apply to bays remote from an edge. However, the deflections and stresses (due to uniform load) in a bay at, or adjacent to, an edge will in general be less than those at the inner bays. The edge bays may, therefore, be safely analysed by the methods to be described. It may, however, be necessary to consider torsional effects which will arise near an edge.

The differential equations governing the deflections and direct stresses in the slab are respectively :—

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q}{D} \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (1)$$

and

$$\frac{\partial^4 f}{\partial x^4} + 2\frac{\partial^4 f}{\partial x^2 \partial y^2} + \frac{\partial^4 f}{\partial y^4} = 0 \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (2)$$

Where additional second order terms, due to membrane action, have been neglected in equation (1). General solutions for equations (1) and (2) are obtained in Fourier Series form which satisfy the conditions of zero deflection and moment at the simply supported ends $x = 0$ and l. These general solutions contain four unknowns, which may be determined by using the following known conditions :—

(a) due to symmetry, the slope $\frac{dw}{dy} = 0$ at $y = \pm\frac{a}{2}$ and 0

(b) due to symmetry, there is no displacement in the y-direction at $y = \pm\frac{a}{2}$ and 0

(c) the equality of strain in the beam and plate at their common surface of contact along $y = \pm\frac{a}{2}$

(d) equilibrium considerations for an elemental length of the beam.

For uniform loading there is rapid convergence of the series solutions for equations (1) and (2), and only the first terms need be taken, this enables the results of the analysis to be expressed in a simple form.

Having solved equations (1) and (2) by using the conditions (a) . . .(d), the stresses, moments and deflections at all points in the beam and slab system may be written down. Fig. 1b shows the bending moments and direct stresses in the slab acting in their positive directions, and also the bending moment and axial force acting at the centroid of the beam cross-section. As these quantities are in general a maximum at mid span $\left(x = \frac{l}{2}\right)$, their values at mid span are given below :—

Mid Span-Stiffening Beam, i.e. $x = \frac{l}{2}\, y = \frac{a}{2}$

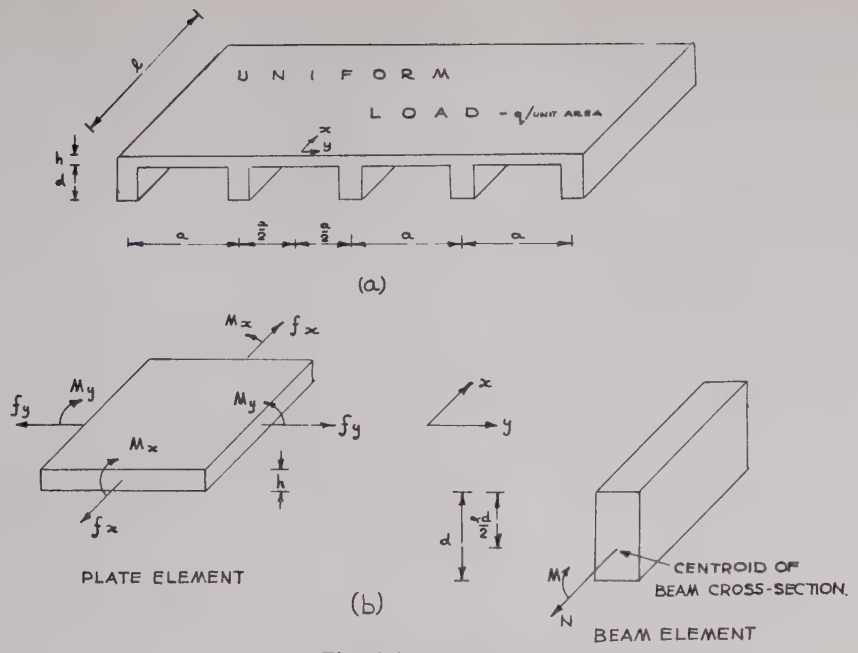

Fig. 1 (a) and 1 (b)

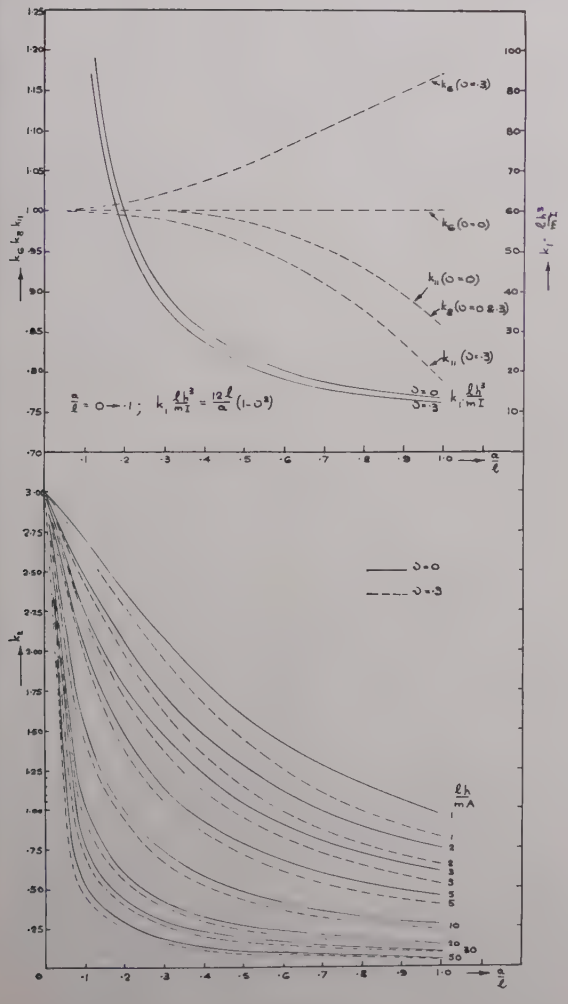

Fig. 2 (a)

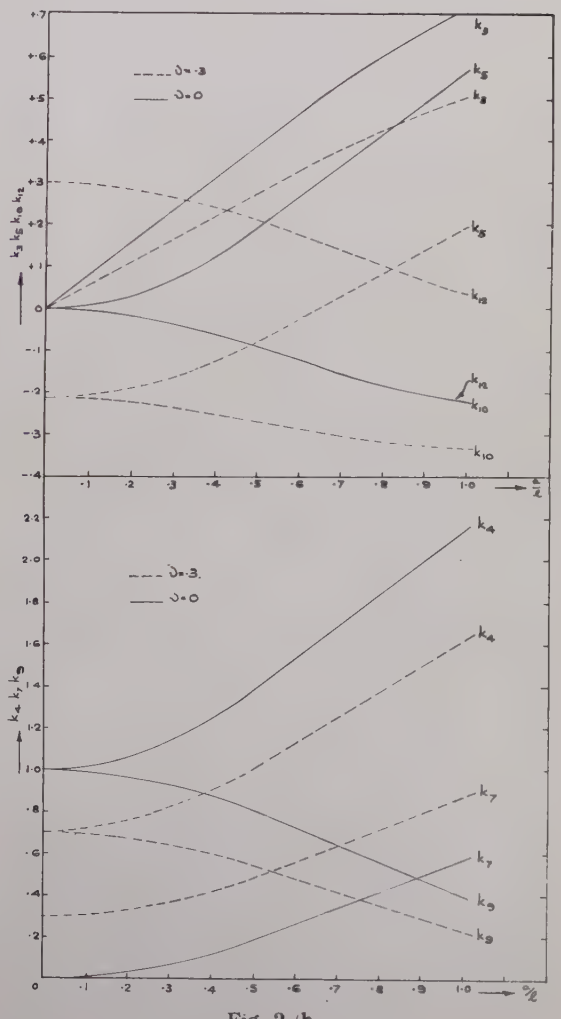

Fig. 2 (b)

$$\text{Deflection} = w = +\frac{5ql^4}{384D} \cdot K$$

$$M = +\frac{ql^2}{8}\frac{EI}{D} \cdot K$$

$$N = +\frac{ql^3}{3h}\left(1 + \frac{\alpha d}{h}\right) K k_2 k_3$$

$$f_x = -\frac{ql^2}{4h^2}\left(1 + \frac{\alpha d}{h}\right) K k_2 k_4$$

$$f_y = +\frac{ql^2}{4h^2}\left(1 + \frac{\alpha d}{h}\right) K k_2 k_5$$

$$M_x = +\frac{ql^2}{8}\left[(K-1)k_6 + 1\right]$$

$$M_y = +\frac{ql^2}{8}\left[(K-1)k_7 + \upsilon\right]$$

Mid Span-Centre of Slab, i.e. at $x = \frac{l}{2}\ y = 0$

$$\text{Deflection} = w = +\frac{5ql^4}{384D}\left[(K-1)k_8 + 1\right]$$

$$f_x = -\frac{ql^2}{4h^2}\left(1 + \frac{\alpha d}{h}\right) K k_2 k_9$$

$$f_y = +\frac{ql^2}{4h^2}\left(1 + \frac{\alpha d}{h}\right) K k_2 k_{10}$$

$$M_x = +\frac{ql^2}{8}\left[(K-1)k_{11} + 1\right]$$

$$M_y = +\frac{ql^2}{8}\left[(K-1)k_{12} + \upsilon\right]$$

where

$$K = \frac{1}{1 + k_1 + k_2\left(1 + \frac{\alpha d}{h}\right)^2}$$

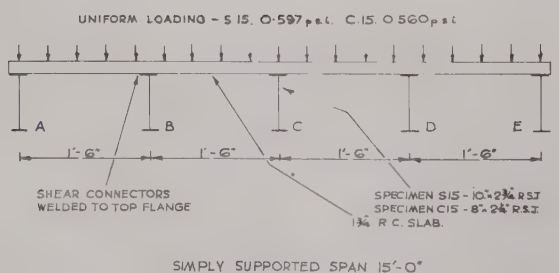

Fig. 3—Transverse cross section

It will be seen that the above quantities are all expressed in terms of the geometrical and elastic constants for the structure, together with a set of coefficients (k) which may be evaluated graphically from Fig. 2. The method is also applicable where the slab and stiffening beams are constructed in different materials.

The equations from which the curves in Fig. 2 were plotted are shown in Appendix I.

Experimental Verification and Worked Example

Results have been published[3] of uniform loading tests on two 15 ft. span beam and slab decks, which may conveniently be used both as a worked example and experimental verification of the theory. The two decks, reference numbers S15 and C15, are shown in Fig. 3, measured deflections and beam strains for both these decks are quoted in reference[3]. It is proposed to show detailed working for S15, and a statement of the results for C15.

Specimen S15

The section and material properties given in reference[3] are reproduced below :—

Beams

Moment of inertia	I	$= 39{\cdot}0$ in.^{4}
Section Modulus	Z	$= 7{\cdot}8$ in.^{3}
Cross-sectional area	A	$= 2{\cdot}64$ in.^{2}
Modulus of Elasticity	E_B	$= 30 \times 10^6$psi
Span	l	$= 180$ in.
Spacing	a	$= 18$ in.
Depth	d	$= 10$ in.

Depth to centroid of beam $= \frac{\alpha d}{2} = \frac{d}{2} = 5$ in.

Slab

Thickness	h	$= 1{\cdot}75$ in.
Modulus of Elasticity	E_S	$= 3{\cdot}8 \times 10^6$psi

Flexural Rigidity per unit width D

$$= \frac{E_S h^3}{12} = 1{\cdot}7 \times 10^6 \text{ lbs/in.}$$

$$m = \frac{E_B}{E_S} = 7{\cdot}9$$

Loading

Load per unit area $q = 0{\cdot}597$ p.s.i.

Hence $\frac{lh^3}{mI} = \frac{180 \times (1{\cdot}75)^3}{7{\cdot}9 \times 39} = 3{\cdot}13$

and $\frac{lh}{mA} = \frac{180 \times 1{\cdot}75}{7{\cdot}9 \times 2{\cdot}64} = 15{\cdot}10$

Reading from the curves of Fig. 2 taking Poisson's Ratio = O,

for $\frac{a}{l} = 0{\cdot}1$; $k_1 \frac{lh^3}{mI} = 120$, $k_3 = {\cdot}08$,

$$\therefore k_1 = \frac{120}{3{\cdot}13} = 38{\cdot}3$$

for $\frac{a}{l} = 0{\cdot}1$, $\frac{lh}{mA} = 15{\cdot}10$; $k_2 = 1{\cdot}2$.

thus $K = \frac{1}{1 + k_1 + k_2\left(1 + \frac{\alpha d}{h}\right)^2}$

$$= \frac{1}{1 + 38{\cdot}3 + 1{\cdot}2\,(1 + 5{\cdot}71)^2}$$

$$\therefore \quad K = 0{\cdot}0107$$

$$\therefore \textit{ Mid Span deflection of beam} = \frac{5ql^4}{384D} \cdot K$$

$$= \frac{5 \times {\cdot}597 \times (180)^4 \times {\cdot}0107}{384 \times 1{\cdot}7 \times 10^6} = \underline{0{\cdot}052 \text{ inch}}$$

$$\textit{Mid Span bending moment on beam} = M = \frac{ql^2}{8}\frac{E_B I}{D} \cdot K$$

$$= \frac{{\cdot}597 \times (180)^2 \times 30 \times 10^6 \times 39 \times {\cdot}0107}{8 \times 1{\cdot}7 \times 10^6}$$

$$= 17{,}900 \text{ in.lb.}$$

$$\therefore \textit{Bending Stress} = \pm\frac{M}{Z} = \pm\frac{17{,}900}{7{\cdot}8} = \pm \underline{2{,}290 \text{ p.s.i.}}$$

$$\textit{Direct force in the beam} = N = +\frac{ql^3}{3h}\left(1+\frac{\alpha d}{h}\right) K\, k_2\, k_3$$

$$= +\frac{{\cdot}597 \times (180)^3 \times 6{\cdot}71 \times {\cdot}0107 \times 1{\cdot}2 \times {\cdot}08}{3 \times 1{\cdot}75}$$

$$= +4{,}670 \text{ lbs.}$$

$$\therefore \textit{Direct Stress in beam} = +\frac{N}{A} = +\frac{4{,}670}{2{\cdot}64} = +\underline{1{,}730 \text{ p.s.i.}}$$

$$\textit{Total Stress in bottom flange of beam} = +2{,}290 + 1{,}730 = +4{,}020 \text{ p.s.i.}$$

$$\therefore \textit{Total Strain in bottom flange of beam} = +\frac{4{,}020}{30 \times 10^6} = +\underline{13{\cdot}4 \times 10^{-5}}$$

These calculated values of deflection and beam strain for S15 are compared in Table I with the corresponding experimental values, the calculated and experimental values for C15 are also included.

It will be seen from Table I that the edge beams, which only support half a loaded panel, have deflections and strains smaller than the calculated values. The agreement between the calculated and experimental results for the central beam (c) is very good.

TABLE I

Strain and Deflections at mid span due to uniform Loading

Specimen	*Beams A and E*	*Beams B and D*	*Beam C*	
S15 Experimental	8·8	11·0	13·5	Strain in bottom flange of beam x 10⁻⁶
Calculated	13·4	13·4	13·4	
C15 Experimental	14·3	16·2	18·4	
Calculated	19·4	19·4	19·4	
S15 Experimental	4·3	4·7	4·9	Beam deflection inch x 10⁻³
Calculated	5·2	5·2	5·2	
C15 Experimental	7·5	7·9	8·1	
Calculated	8·6	8·6	8·6	

Although experimental strains in the slab are not quoted in reference[3], the stresses in the slab may be calculated as follows.

At the mid-span the longitudinal bending moment per unit width of the slab is given by :—

$$M_x = \frac{ql^2}{8}\left[(K-1)\,k_6 + 1\right] \text{ and for } \frac{a}{l} = 0{\cdot}1,\ k_6 = 1$$

$$= \frac{{\cdot}597(180)^2}{8}\left[({\cdot}0107 - 1)1 + 1\right]$$

$$= 25{\cdot}9 \text{ in.lb.}$$

The slab is lightly reinforced longitudinally, hence

$$Z \simeq \frac{h^2}{6} = 0{\cdot}51 \text{ ins.}^3.$$

Therefore the longitudinal bending stress

$$= \pm\frac{M_x}{Z} = \pm\frac{25{\cdot}9}{{\cdot}51} = \pm\underline{51 \text{ p.s.i.}}$$

The longitudinal direct stress at mid span is,

$$f_x = -\frac{ql^2}{4h^2}\left(1+\frac{\alpha d}{h}\right) K\, k_2 k_9, \text{ and for } \frac{a}{l} = 0{\cdot}1,\ k_9 = 1$$

$$= -\frac{{\cdot}597(180)^2}{4(1{\cdot}75)^2}\ (1 + 5{\cdot}71) \times {\cdot}0107 \times 1{\cdot}2 \times 1$$

$$= \underline{-136 \text{ p.s.i.}}$$

Hence the total stress at the top of the slab

$$= -136 - 51 = -187 \text{ psi} \Big\} \text{ compressive}$$

and the total stress at the soffit of the slab

$$= -136 + 51 = -85 \text{ psi} \Big\} \text{ compressive}$$

References

1. Hendry, A. W., and L. G. Jaeger, "Grid Frameworks and Related Structures," (Chatto and Windus Ltd.).
2. Holmes, M., "Stiffened Plating under Transverse Load," *Quarterly Journal of Mechanics and Applied Mathematics*, Vol. 12, Part 4, November, 1959, p. 443-453.
3. Studies of Slab and Beam Highway Bridges Part 1. *University of Illinois Bulletin* 363. Newmark, Siess and Penman.

Acknowledgement

The Author wishes to express his appreciation of the advice given by Professor Sir Alfred Pugsley during the progress of the work described in this paper.

Appendix I

$$k_1 = \frac{3\pi(1+\lambda)^2}{\tanh\frac{\pi a}{2l}} \cdot \frac{ml}{lh^3}$$

$$k_2 = \frac{3(1+\lambda)}{3 - \lambda + 4/\pi\,\frac{lh}{mA}\tanh\frac{\pi a}{2l}}$$

$$k_3 = \frac{(1-\upsilon)\tanh\frac{\pi a}{2l}}{(1+\lambda)}$$

$$k_4 = \frac{\left(\frac{3+\upsilon}{1+\upsilon} - \lambda\right)(1-\upsilon^2)}{(1+\lambda)}$$

$$k_5 = \frac{\left(\frac{1-\upsilon}{1+\upsilon} - \lambda\right)(1-\upsilon^2)}{(1+\lambda)}$$

$$k_6 = \frac{2\upsilon}{1+\lambda} + 1 - \upsilon$$

$$k_7 = \frac{2}{1+\lambda} - (1-\upsilon)$$

$$k_8 = \frac{1+\lambda\cosh^2\frac{\pi a}{2l}}{(1+\lambda)\cosh\frac{\pi a}{2l}}$$

$$k_9 = \frac{\left(\frac{3+\upsilon}{1+\upsilon} - \frac{\pi a}{2l}\coth\frac{\pi a}{2l}\right)(1-\upsilon^2)}{(1+\lambda)\cosh\frac{\pi a}{2l}}$$

$$k_{10} = \frac{\left(\frac{1-\upsilon}{1+\upsilon} - \frac{\pi a}{2l}\coth\frac{\pi a}{2l}\right)(1-\upsilon^2)}{(1+\lambda)\cosh\frac{\pi a}{2l}}$$

$$k_{11} = \frac{1+\upsilon+(1-\upsilon)\frac{\pi a}{2l}\coth\frac{\pi a}{2l}}{(1+\lambda)\cosh\frac{\pi a}{2l}}$$

$$k_{12} = \frac{1+\upsilon-(1-\upsilon)\frac{\pi a}{2l}\coth\frac{\pi a}{2l}}{(1+\lambda)\cosh\frac{\pi a}{2l}}$$

$$\lambda = \frac{\frac{\pi a}{2l}}{\sinh\frac{\pi a}{2l}\cosh\frac{\pi a}{2l}}$$

A Commentary on the New British Standard 449 (1959): The use of Structural Steel in Building *

Discussion on the Paper by John Mason, B.A.(Cantab.), M.I.Struct.E., A.M.I.C.E. (Hon. Treasurer)

THE PRESIDENT proposed a vote of thanks to Mr. Mason, first for having taken the trouble to write his paper in time for it to be published in THE STRUCTURAL ENGINEER so that members could read it before coming to the meeting, and secondly, for his clear exposition of certain of its salient points.

Discussion

MR. R. GRAY (Member), said he had submitted a written question on Clauses 13 and 14 which related to combined stresses. He had noticed that in the new edition of the Specification careful reference was made to wind stresses and in the combined formula given in Clause 14 an increase in stress was permissible only in relation to those stress ratios which actually included wind stresses. This led to an extraordinary position as indicated by the figures which he had submitted to Mr. Mason, and he would be glad to have his comments.

MR. R. G. TAYLOR (Member), said that the part of the Specification which interested him particularly was the introduction of the 2½ per cent rule covering the requirements for lateral stability of compression.

Clause 31 referred to specific reductions of the effective lengths of discontinuous struts from 0·7 to $0{\cdot}85\,\frac{l}{r}$. The provisions were not intended to apply to continuous angle struts which it was stated should be designed in accordance with Clause 26. Referring to Clause 26, it was found that the effective length of all struts was the maximum length between the points of effective lateral support. To be effective, these points of lateral support had to be capable of providing the 2½ per cent resistance required by the standard. In other words, the continuous strut was deemed to be less effectively restrained than the discontinuous angle strut since no reduction of length was permitted even where joints were restrained.

If the rafter of a roof truss was designed between purlins, then the purlins, in addition to carrying their normal load, would also have to carry a proportion of the 2½ per cent resistance required. In that case, how did one design a purlin since the purlin rule was empirical and not related directly to stress in the purlin.

If the points of lateral supports were not purlins, then they were presumably some form of restraint connected to roof bracing capable of providing the 2½ per cent resistance and in that event, the rafter would have to be designed on the full length between the points connected to the bracing. We should, therefore, in future have to design our rafters on much longer panel lengths than in the past or supply more lateral bracing or design purlins on combined stress and then connect them to bracings.

As a seeker after the truth, he asked where the 2½ per cent came from and suggested that in the overthrow from previous specifications, it was put in more in ignorance than for any other reason. Would not 1½ per cent do? He would like to know why there was a new decision to consider a cumulative 7½ per cent and how was it that this had been fixed at a limit.

Another aspect was the separation of loadings, including wind, from B.S.449. On the face of it, it seemed a harmless and indeed praiseworthy thing to do. One had to be careful, however, in using a separate and arbitrary loading specification. In 1937, there were certain wind loadings and in 1948 these were altered to more realistic figures. He understood that wind loadings might now be increased and brought more into line with American practice. Should American figures be adopted, then even on small buildings of say 10 ft. to 20 ft. high, the loading due to wind, especially with internal pressure, would be far greater than the dead plus imposed loading. Even allowing for the 25 per cent increase in stress allowance for wind loadings with our present standard, we should then have the undesirable state that the bottom tie of a roof truss would work out greater than the rafter. In other words, although the wind loadings might be realistic, their application to B.S.449 might well produce unrealistic results. B.S.449, therefore, should be reviewed and revised simultaneously with any revisions to Chapter 5 to keep it in line with reasonable practice. If this was not done, then due to the alteration in wind since 1937, most structures where reversals occurred, would be unsafe.

MR. D. A. TAUNT (Associate-Member), said the new B.S.449 specified a minimum thickness of metal for steel tubes of 10 gauge. Manufacturers' handbooks however, showed that the standard thickness of many useful tubes was 9 and 11 gauge but not 10 gauge.

* *Read before the Institution of Structural Engineers, at* 11, *Upper Belgrave Street, London, S.W.*1., *on the* 26*th November*, 1959. *Mr. L. E. Kent, B.Sc., M.I.Struct.E., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer," Vol. XXXVII, No.* 11, *pp.* 335-342 (*Nov.* 1959).

He asked if there was any tie-up with the manufacturers which might lead to their producing sections conforming with this ' minimum thickness ' standard.

Mr. G. B. Godfrey (Associate-Member of Council), observed that the new edition of B.S.449 was noteworthy in that for the first time steel tubes and rectangular hollow sections had taken their place alongside conventional rolled steel sections. Mr. Mason had mentioned the five types of tubes which were allowed by the new Standard but, in fact, practically all the tubes employed for structural purposes were either hot-finished welded tubes made by the continuous weld (Fretz-Moon) process or hot-finished seamless tubes made by the rotary forge (Pilger) process.

Grade 13 tubes (yield strength 13·5 tons per sq. in.), were allowed but as their cost per ton was only a little less than Grade 16 tubes (yield strength 16 tons per sq. in.), the great majority of mild steel tubes used in structures were in the higher strength steel. High tensile steel tubes of Grade 20 were specifically mentioned, but steels of higher yield strength were also available.

Clause 12, minimum thickness of metal, had been mentioned in the discussion. It was of interest to note that the wall thickness of tubes could be less than 10 gauge if some special protection against corrosion (such as galvanising) were provided.

Mr. Godfrey particularly wished to draw the attention of members to Mr. Mason's remarks in the section entitled *Basic Stresses* on p.338 of his paper. His own impression had been that higher stresses were allowed in flexure because they were localised in the outermost fibres of the member, whereas the lower stresses allowed in tension or compression were often uniformly distributed over the whole section. It was, however, perfectly logical for Mr. Mason to invoke the Plastic Theory to justify the bending stress of 10·5 tons per sq. in. allowed for rolled steel joists and channels, their shape factors being about 1·15 and 1·17 respectively, or 10 tons per sq. in. for plate girders, as their shape factor was usually somewhat lower. The shape factor for most structural tubes, however, exceeded 1·30, increasing with the thickness/outside diameter ratio and reaching 1·70 for a solid bar. This was known to Dutch[1] and German [2-3] dock engineers who used the guaranteed yield stress (in tension) as the allowable working stress in the high tensile steel tubes employed in dolphins.

The load factor for tubes and hollow sections would be no less than the value of 1·75 for an R.S.J. if the allowable bending stresses for Grade 16 and 20 material were raised to 11·5 and 14·0 tons per sq. in. respectively. Mr. Godfrey believed that they could be even higher for seamless tubes as the manufacturing processes resulted in the yield strength being much higher than the values implied by the relevant Grades. Would Mr. Mason kindly comment on this reasoning ?

Mr. Godfrey emphasised that Tables 3 and 4 in B.S.449 did not apply to tubes or hollow sections, as flange instability would not occur.

He then sketched the curve for mild steel or Grade 16 struts derived from Table 17 of the Standard, as shown in Fig. A1.

He stated that although the buckling of a slender strut (i.e. in the Euler range) was independent of the profile of a member, the effect of shape on a stout column was most marked, a tube being much stronger than say, an R.S.C. or an angle. Unfortunately, only the Germans had officially recognised this fact. Until 1959, the curve for all stout mild steel struts, derived from the appropriate table in DIN 4114, had been as shown in Fig. A1. In January 1959, however, the German building authorities agreed to raise the stresses for stout tubes, with the proviso that the maximum allowable stress should not exceed the basic stress of 1,400 kg. per sq. cm. (8·89 tons per sq. in.). The relevant research and justification for the increased stresses were contained in a paper by Klöppel and Goder[4].

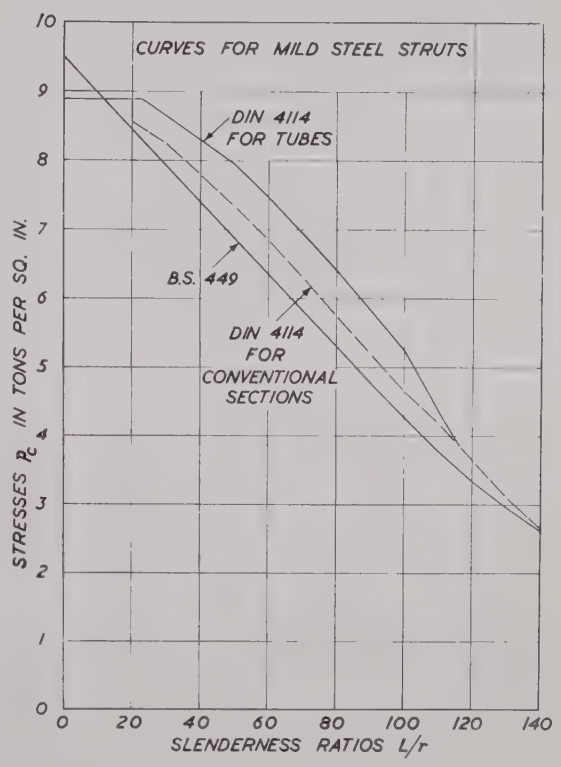

Fig. A1

It appeared that there was more than a reasonable case for a similar increase in the allowable stresses for stout tubular struts here. Perhaps this could be considered during the periodical review of the Standard.

Mr. Mason had quoted the empirical formulas for angles used as purlins. Mr. Godfrey observed that there were also formulas favourable to tubes.

Mr. J. Ford (Associate-Member) said that the same use of increased stresses in filler joist floors was permitted in this specification as in the previous one and he presumed that this was intended to take account of composite action between concrete and steel.

He felt that in the absence of any warning to the contrary, there was a danger that this advantage might be taken when in fact it was not valid. Should a contractor elect to suspend the shuttering from the floor joists without any other support, the joist alone would carry the dead load and the composite section only the live load. Such a procedure could result in the joist being quite severely overstressed if the full live load were applied, and he suggested that some warning should be given so that the designer would be reminded to specify the support of joists when the floors were cast.

Mr. L. E. Ward (Member of Council), referring to the semi-rigid method of design, asked if it would not be possible, inasmuch as the 1936 recommendations had been republished as an addenda to the present document, to modify it so that it would be at least a little nearer to the stresses used in design now.

Secondly, in regard to purlins, no differential seemed to have been made, whether a purlin was simply supported or continuous. We all appreciated that the deflections in a continuous purlin were half as much as when it was simply supported.

Mr. G. Hogan (Associate-Member), following Mr. Godfrey's remarks on tubes, asked if the position concerning the l/r ratio and the relevant D/t column (Table 3) to be used when designing a tube as a beam, could be made clearer.

Mr. Ward commented that Table 8 in B.S.449 related to a critical stress and an allowable working stress, which were literally taken from B.S.153, and the factor of safety seemed to vary from 2 to 8. It appeared that, as a girder became stronger, so the factor of safety increased many times. The paper referred to this and to an allowance for imperfections, but he could not think of more than the critical stress and the allowable stress. We ignored that in B.S.153 ordinary rolled beams were designed under the same formula.

Concerning combined stresses, he said that in cased columns and cased struts we had to divide the actual load by the allowable load. He asked if that would lead to rather revolutionary designing, and suggested it would be simpler if, instead of designing our concrete to a stress of 830 lb/sq. in., we added 1/30th to the area and divided by the permissible stresses in the usual way.

Mr. John King (Associate-Member) remarked that, having looked at the new document and listened to all the very erudite remarks made at the meeting, he had concluded that the document was tricky, to say the least. He wondered what the Author thought about the point that, whereas it was probably quite proper to have the document in the hands of senior engineers, it must be remembered that it would be used by unqualified men, often fairly young men in the office. If qualified men had difficulty in understanding and interpreting everything in the Standard, how much more difficult would it be for the younger men who had to use it every day.

Mr. C. M. Dean (Graduate), pointed out that the new B.S.449 made no mention of angles in bending. In formula (2) (page 339 of the paper) certain factors were given relating to I sections, and he asked if Mr. Mason could indicate how we could modify the formula to fit short angle sections.

Referring to bending in heavy compound columns, which might be made of three rolled steel joists, one forming the web with the others attached to each flange, one could obtain quite sturdy sections to which the code did not seem to give full coverage. We could not, according to the code, take into account the favourable D/t ratio given by considering the whole of the rolled steel joist forming the flanges in order to increase the average value of T. He thought that the rolled steel joist forming the flange tended to stiffen the member rather more than the code permitted and he asked why provision was not made in the British Standard to make allowance for such sections.

Mr. O. A. Kerensky (Member), discussing wind loading, said that the intention of the Specification was that every structure should in the first instance be designed to carry dead loads and live loads at normal allowable stresses and then checked for wind and secondary effects of live load, if any, using appropriately increased working stresses. He did not see any room for controversy in this respect and maintained that if in any member there were any increase in stresses due to wind, whether bending and/or axial, all the allowable stresses should be increased by 25 per cent, that is :

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \not> 1{\cdot}25$$

If the above equation was satisfied when stresses due to wind were included, the make-up of the member need not be altered, if not, it must be modified in such a way that the equation was satisfied. That was the method specified in the current B.S.153 but not in the latest B.S.449. He hoped, however, that this would soon be amended.

With reference to the strength of tubes in compression, the Perry-Robertson formula was based on tests carried out on joists and rectangular sections, and it was true that tubular sections were superior to either. The strut formula contained no shape factor, whereas the new beam formula did. He suggested that higher average compression stresses could be adopted for tubular struts than for the less favourable sections, such as I beams, angles and channels. He doubted if a tube in bending would ever fail through lateral instability and believed that most tubes in bending would in fact be designed against failure by yield, in other words, at maximum allowable bending stresses.

He regretted that in the latest B.S.449 no mention was made of angle and Tee sections in bending. For example, a Tee acting as a slender beam was obviously more stable laterally when the horizontal table was in compression than when it was in tension. The B.S.153 rules reflected this fact. For any given l/r, the critical stress for a Tee with its table in compression was greater than, and with its table in tension smaller than, that for a corresponding I section. Dealing with crane gantry girders, in which channels or joists, placed horizontally, were used as compression flanges, the new beam formula reflected the improvement in the lateral stability of such sections by taking into account the shift of the neutral axis towards compression flange and the improved slenderness ratio of the section as a whole.

Mr. Kerensky could not understand a suggestion by one of the speakers that the load factors in the new beam formula varied from 2 for long slender beams to 8 or more for short ones. The allowable stresses in short beams were related to yield stress of the material with a safety factor of about 1·5, and in long beams to critical buckling stresses with a load factor of about 2. There was an intermediate transition zone where the critical stress curve was modified to merge into the yield stress, and the load factor in this region could be as small as 1·4. A load factor of 8, or even of infinity could be obtained by comparing the allowable stresses with the theoretical critical stresses as given by the critical stress curve (similar in shape to the Euler curve for struts). At slenderness ratios of somewhere between 60 and 100 this curve became asymptotic and, therefore, could bear no relation to the actual strength of the compression flange.

Mr. P. L. Capper, T.D. (Member), asked if Mr. Mason could explain a point in connection with formula (2) (p. 339), where he had referred to substituting some approximate geometrical properties, among which was :

$$\gamma = \frac{I_x - I_y}{I_x} = 1$$

He commented that that quantity could not be 1.

Recalling Mr. King's comment that a number of tricky points had been included in the new Specification, Mr. Capper considered it was a very good thing to do so. The policy in regard to some documents of that type was to dispose of matters which were somewhat controversial by leaving them out.

Mr. Ward, referring to Fig. 2, said the load factor curve was not midway between the base line and the critical stress curve until it reached 10. He considered that the curve could be higher; but he did not know how high.

Mr. C. S. Gray (Associate-Member), pointed out to the meeting that the new B.S.449 was not a text book, it still left a little room for the exercise of engineering ingenuity; that was why some of the things in it might appear to be liberal.

On the question of what to do with angles, he said they did not suffer a reduction factor and therefore, they took the full stress. Taking the document as it is, it meant exactly the same thing, $1\frac{1}{2}$ or whatever it might be, according to the strength of the material.

Concerning bending members, if one took the trouble to read B.S.153 one found the magic quantity:

$$\frac{I_x - I_y}{I_x} = 1$$

which covered everything. If I_y were bigger than I_x, probably the formula did not apply at all. The formulae in the document were based on a lot of fundamental research, and we could rely on them.

Mr. Taylor speaking of purlins, said that with the introduction of the 15 lb. imposed loading against 10 lb. previously applied, it was realised with horror that almost every purlin in the country would be increased in size. The rule was altered to $WL/115$ instead of $WL/90$ previously used so that the purlins would not alter with the new imposed loadings. He considered this reasonable.

Unfortunately, when it came to bringing tubular purlins in line with these modifications, it could not be done since previously they had been designed on theoretical rules of $WL/80$ and $WL/120$ implying simply supported or continuous purlins at 10 tons per sq. in. To overcome this difficulty, tests were carried out on a 27 ft. 6 in. span roof, 75 ft. long with trusses at 15 ft. centres and with 4 in. × 3 in. × $\frac{5}{16}$ in. angle purlins on one slope and 3 in. o.d. × 10 ga. tubular purlins on the other. Tests were made on semi-continuous purlins, on purlins continuous over two bays with connections similar to rolled steel angles and finally, on discontinuous purlins with a single bolt connection at each end to simulate what might loosely be regarded as simply supported conditions.

A special bridge to match the roof slope had been built so that accidental loads would not be applied to the roof during tests and increments of loading had been applied taking all tests up to $22\frac{1}{2}$ lb. imposed load. The tests showed that although the deflections under these severe loads were visible, even under the worst conditions, there was no permanent set and no damage to sheeting or fastenings. The results of these tests supported by practical experience form the basis for design rules in B.S.449 for tubular purlins and confirm the rules for angle purlins.

Mr. W. W. Barker (Associate-Member), said that in two places in the new British Standard mention was made of deflection, one in connection with elastic theory in simple design and the other plastic theory in rigid design. He asked if Mr. Mason would adopt normal theory for calculation in both cases, or make some modification in terms of a plastic modulus or inertia, for the latter design conditions.

Further, the Code did not appear to allow for the assistance given in many cases by construction, the torsion condition in beams and connections for example, would disappear if the construction was considered. To design buildings as steel framed free-standing skeleton frames was still common practice and the assistance given by construction was ignored. The bending moment distribution in a rigid frame would be much affected if construction was considered. Wall beams, for example, could be designed as deep beams to cill level, and floor slabs would assist beams in bending.

Thirdly, Clauses 21 and 30 contained regulations for concrete encased steel beams and stanchions, giving working stresses, radii of gyration, and equivalent stanchion areas etc., but did not include rules for determining the stiffness factors, or inertia values, when using these sections in continuous or rigid structures. These members were heavily reinforced and could easily be integrated with main structure if the building had walls and floors.

Mr. Mason, replying to the discussion, first thanked the speakers for their interesting, even if difficult, questions. He also expressed appreciation to Mr. C. S. Gray for having said what he himself should certainly have said, and which he now repeated, that when all was said and done we had not all the answers to all the questions automatically on some page or other of B.S.449; it was a guide and perhaps a rule, but it still left a lot for the Engineer.

The question raised by Mr. R. Gray, both verbally and in writing, had been answered by Mr. Kerensky; and whilst thanking him for doing so, Mr. Mason added a little more. The difficulty, he said, was in the interpretation of Clause 14, in relation to Clause 13 and related to the two factors which must not exceed 1, i.e., the ratio of the direct actual stress to the direct permissible stress and the ratio of the actual bending stress to the permissible bending stress. Mr. Gray had written that if there were no direct stress due to wind we could not increase the $\frac{fc}{pc}$ factor. But the specification stated 'ratios which include wind,' and Mr. Mason said he would not have much difficulty in convincing himself that both those factors included for wind and could be increased. After all, when we had 15 lb. on a roof it included for snow, we did not say 'in an area where there actually is snow.' He would read Clause 13 as including wind.

Mr. Mason, coming to the question concerning the $2\frac{1}{2}$ per cent which Mr. Taylor regarded as being adequate to give lateral support, said he supposed it was empirical, and we could not argue with empirical rules; there were still a lot of them in our specifications. He was brought up to take $2\frac{1}{2}$ per cent for that purpose and he had been with it for so long that he almost accepted it; it had perhaps a little theoretical justification. If we took the Perry-Robertson allowance for imperfections and put in a lateral force sufficient to balance that out, we should find that it came to about $2\frac{1}{2}$ per cent.

Then there was the question that continuous rafters seemed to be at a disadvantage compared with discontinuous members. Indeed they were. This was a point which had worried him quite a lot, but he did not know the answer to it. A free ended strut would

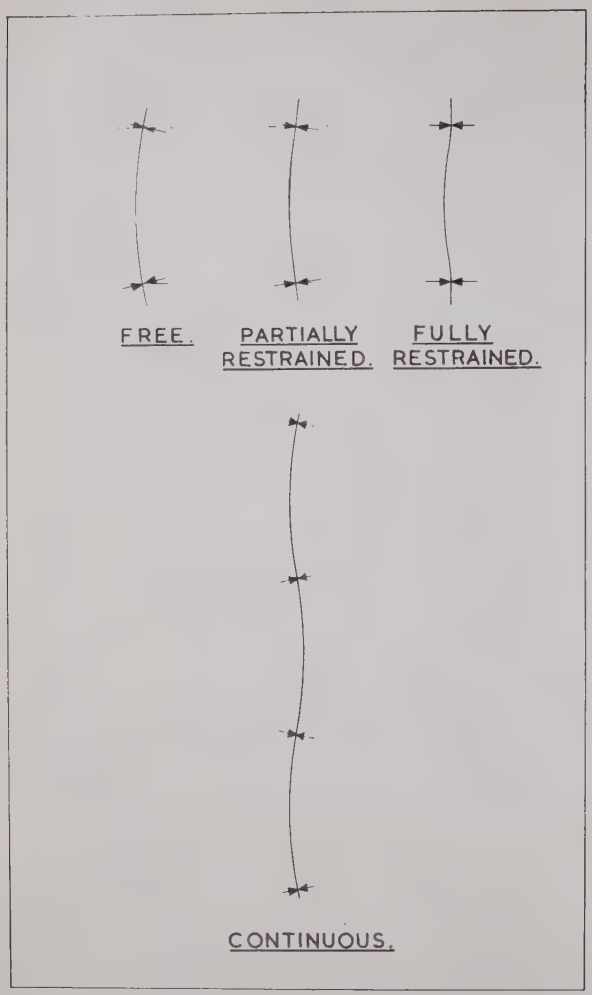

Fig. A2

buckle in single curvature — see Fig. A2 — a fully restrained strut would buckle in triple curvature; a partially restrained strut would also buckle in triple curvature.

If a continuous strut were held in position only at the two nodes, there was no doubt that it would buckle in single curvature. It seemed strange, but it was so.

With regard to tubes, Mr. Taunt had raised a point on the tie-up with the British Standard. Mr. Godfrey was very largely responsible for representing the tube interests on the British Standards Committees; he had dealt with this question so well that one felt he had answered the point.

As to whether the shape factor is greater for tubes than R.S.J. and that therefore the apparent permissible stresses in tubes could be greater than in rolled steel joists, he thought that probably they could be. He agreed with Mr. Kerensky's remarks on that matter. The form factor for a joist was about 1·15 and if in fact the form factor for a tube was greater than for a joist he did not see why it should not be taken. There might be other considerations coming into it. These matters were often quite complicated, even though they appeared to be simple. He could not understand why Mr. Godfrey had not put this point to the British Standards Committees.

The question of filler joists was raised by Mr. Ford. Generally speaking, said Mr. Mason, we had now reached a stage in stresses when the deflections were becoming more important. Deflection was creeping in as a criterion, and Mr. Ford had mentioned a real difficulty. In the case of a floor, be it of filler joists or, as he would prefer, of reinforced concrete, its inherent strength, as it appeared on test, was very great, greater than was assumed in our calculations; although he would not be a bit worried about the floor when it was finished, one might find that there was a deflection when the weight of the concrete came on to it.

A very interesting point was raised by Mr. Ward, one which was very much in the forefront of Mr. Mason's mind when writing that part of the paper, concerning the Recommendations for semi-rigid design which were made in the third of the Steel Structures Reports, in 1936. Mr. Mason said that in the paper he had made the point that stresses had increased but that anyone who was designing to those Recommendations was working to stresses which had not been so increased and was therefore at a disadvantage. Mr. Ward would have noted that the British Standard Specification permitted one to sit down and re-write those Recommendations with modern stresses in them; but the difficulty was that they were a set of Recommendations complete in themselves, and they made in total one formula. We could not tinker with just one factor in a formula unless we also considered all the other factors; an enormous amount of work went into that formula, and we could not just alter one part of it. Whilst he sympathised with Mr. Ward, Mr. Mason considered that the only thing to do was to re-write the Steel Structures Reports Recommendation which was permitted.

The design of purlins was rule-of-thumb, and again it was empirical; it did not lend itself to analysis of a mathematical or elastic kind. Mr. Taylor had dealt with that very fully. Mr. Mason said that in his own practice he would be glad to see some tie rods used to take out some of the deflections; apart from that, we had simple Rules and they worked.

He was not quite clear whether Mr. Hogan was referring to a single tube or to tubes made up into a girder, but he hoped Mr. Kerensky had answered the point.

Then Mr. Ward had referred to critical stresses and an apparent factor of safety of from 2 to 8, and Mr. Kerensky had answered that. The particular point could be illustrated by the Perry-Robertson formula for struts.

The Graph (Fig. A3) plots the Euler critical load and the yield stress. If we had a column pin-jointed and we loaded it, and if it were a very slender column, there was a critical load at which quite astonishing bending occurred, but it would spring back to its original form. That could easily be demonstrated with a T-square. But another type of mechanism came into play at a point where the yield point was reached. The Perry-Robertson formula married the two curves by assuming that the imperfections were equivalent to an initial deflection of $0{\cdot}003\ l/r$, and when that was worked into a formula we obtained a curve which ran out at the Euler load for high l/r factors i.e., the Perry-Robertson ultimate in the graph and a factor of safety was placed on that. If, however, the intermediate stage is left out and the working load is compared directly with the critical load then the apparent factor of safety rises from 2 to a much higher value.

It was said by Mr. King that the specification was tricky. Indeed it was.

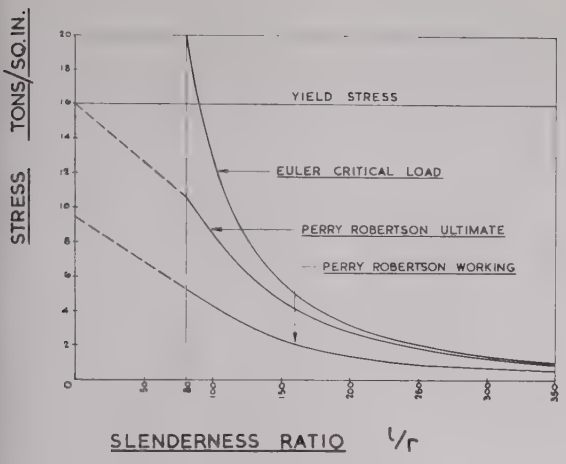

Fig. A3

The points raised by Mr. Dean were covered by Mr. Kerensky, who had been very kind; he had not asked awkward questions, as one was sure he could have done had he wished, but he had been very helpful in answering a lot of the speakers.

Reference was made by Mr. Capper to the substitution of approximate geometrical properties in formula (2), on page 339. He would be the first to realise, however, that formula (2) was almost unmanageable. Basically it was for a beam where the moment of inertia about the $x - x$ axis was greater, very much greater, than the moment of inertia about the $y - y$ axis. Therefore, for the purpose of simplifying formula (2) the value of γ i.e. :

$$\frac{I_x - I_y}{I_x}$$

was taken as being 1. Mr. Capper might say that that was not a very good approximation. It had been so taken, however, in order to get formula (2) into a manageable form.

Three very interesting questions were raised by Mr. Barker. Mr. Mason repeated his earlier remark that deflection was becoming a problem today. The British Standard stated that : " Alternatively, a fully-rigid design may be based on the principles of the plastic theory so as to provide an adequate load factor and with deflections under working loads not in excess of the limits implied in this British Standard." That should enable the engineer to do pretty well all that he wanted to do. Perhaps the only helpful thing that one could say was that in a rigid design the fixing of the ends of the beams did tend materially to reduce the deflections, as compared with a free-ended beam.

On stiffness factors the whole thing was very approximate. All that we could hope for was that the *ratio* of the stiffness of one member to another which was what governed the distribution of stress was not too far wrong.

These were all difficult points which he did not think we could answer in a document such as the British Standard. So that we came back to where we started; as Mr. Gray had said, there was still a lot left for the Engineers to do. The document might give some help, it provided something by which to measure our work, but we certainly did not get all the answers out of it. Certainly anybody who was not an Engineer, and into whose hands we put this document, could not possibly imagine that he could do the Engineer's work for him.

Finally, Mr. Mason thanked the meeting very much indeed for the way they had received his paper and for the most interesting questions they had put to him.

THE PRESIDENT, at the close of the discussion, said that, strictly speaking, he should ask the meeting to thank Mr. Mason for the adequate way in which he had answered the questions put to him. But one was not certain that one should not also include Mr. Kerensky and Mr. Gray in that remark.

However, the paper had come at a very opportune moment, because at 10.30 on the following morning, members of B/20 committee would be considering the document again, and no doubt some of the points raised at the meeting would be further discussed.

B.S.449 was a document which had to be considered with a certain amount of sense, horse sense. The B.S.I. Committee had a Chairman, Mr. G. A. Gardner, who guided the members of the committee during the production of this document, and he had a theory that B.S.449 should consist of one sentence only—" Every structure should be designed by a competent engineer."

At the moment, however, the President said it was his pleasure to ask the meeting to accord Mr. Mason a second vote of thanks for the way in which he had answered the questions.

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The Collapse of Triangulated Trusses by Buckling within the Plane of the Truss*

Written discussion on the paper by J. G. Nutt, B.E.

Mr. M. Gregory congratulated the Author on his paper, and referred to existing methods for calculating the critical load of a triangulated framework, as mentioned in the introduction. That critical load was the buckling load for the "mathematically perfect" structure, assuming perfectly straight members, no eccentricity at the joints, and no yielding. The problem was to relate the behaviour of the practical structure having practical imperfections such as initially crooked members, eccentricities at joints, and having a finite yield strength, to the mathematics of the perfect structure, and, in particular to find the collapse load of the practical frame. The collapse load of a light flexible frame made of material of high yield strength might be close to the calculated critical load, but unfortunately, for structures containing members of stiffness in the practical range, this was not the case. Therefore, any attempt to tackle the problem of studying the behaviour of practical frames proportioned in a realistic manner was worthwhile. Some method of taking account of the practical imperfections of framed structures was urgently needed.

The method of limiting the load-deflection relation for a frame by the elastic stability line and the plastic collapse line was very useful. Further experimental work such as indicated in Fig. 14 of the paper, should furnish a means of estimating the collapse load of frames by expressing the reduction of the collapse load below the intersection of the plastic collapse line and the elastic stability line as a function of the yield and critical loads.

Mr. Nutt's method therefore depended basically on the drawing of the plastic collapse line and the elastic stability line. The former was defined by his equation (1a)

$$\Delta = \frac{2M'}{P} - \frac{P}{P},$$

and was easily calculated. It was the elastic stability line to which Mr. Gregory's remarks were directed. It could be seen from Figs. 7-13 that accurate drawing of that line was important, and this particularly applied to the more slender frames.

The elastic stability line was defined by equation (2)

$$\Delta = \frac{a + e}{1 - \frac{W}{W_c}}.$$

In plotting this line, Mr. Nutt had measured the initial crookedness a, measured joint eccentricities and calculated the effective eccentricity e for the member, and calculated W_c by the moment distribution convergence criterion, or Allen's approximation. Figs. 7-13 showed the validity of his measurements and calculations. However, in order to apply this method to actual frames in practice, there must be a means of obtaining values of a, e, and W_c. As Mr. Nutt stated, the initial imperfections of the frames tested were actually measured, but this was not possible except in research trusses, and an assumed value of eccentricity must be used if the method was to be used in design. Mr. Nutt suggested the value $0{\cdot}003\, l/r$, which was the

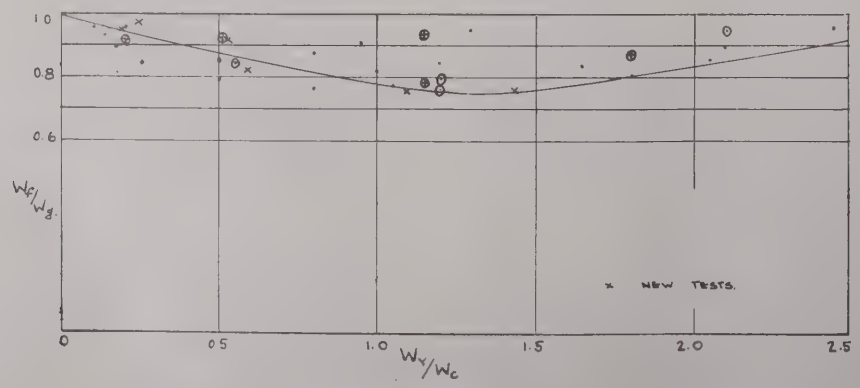

Fig. 14

* Published in "*The Structural Engineer*," Vol XXXVII, No. 5, May, 1959.

BS449 value for the initial crookedness of a practical pin-ended strut. Professor J. F. Baker's comments in Chapter 2 of "The Steel Skeleton," Vol. I, were of interest in this connection. In fact only very limited information was available on the likely crookedness and eccentricity of struts in frames.

Mr. Gregory commented that the Southwell Plot furnished a very powerful experimental method of obtaining values of $(a + e)$ and W_c for any elastic structure liable to instability and suggested that Mr. Nutt might carry out the Southwell Plot on his deflection measurements shown in Figs. 7-13. It was necessary only to plot Δ/W against Δ as far as first yield. It would be interesting to find how the reciprocal of the slope of the plot compared with W_c, and its intercept with $\frac{a+e}{W_c}$, as the equation of the plot was

$$\frac{\Delta}{W} = \frac{\Delta}{W_c} + \frac{a+e}{W_c},$$

in the elastic range. This furnished an accurate check on the measured and calculated values of $(a + e)$ and W_c, and, incidentally, a method of obtaining their values in more complicated cases where they could not be calculated or measured.

In the long run, for practical application of Mr. Nutt's method, the statistical distribution of $(a + e)$ and the value of W_c would be required for all types of structures. The use of the Southwell Plot on deflections was a convenient way of obtaining this information.

However, Mr. Gregory found the Southwell Plot on *measured strains* much more powerful. It was possible to show mathematically that that plot was linear for the case of a pin-ended strut and also for a triangular frame. He had checked this experimentally and obtained good agreement. He had also carried out measurements on web members of lattice girders, bolted angle members in frames, and the compression chord of a lattice girder as it buckled laterally, and obtained linear Southwell Plots on strains.

Strains were convenient to measure, and by this means a great deal more information could easily be obtained. In certain cases, where a strut in a structure was not too stiff, the equation of the Southwell Plot on strains could be used as a *direct collapse criterion*. If there was not too much reserve of strength beyond first yield, the maximum strain could be put equal to the yield strain. Solution gave the collapse load.

He had made the following analysis of Mr. Nutt's results :—

TABLE 4

Code No.	*Type of Frame*	*L/r*	*Collapse Load*	*Load to Cause First Yield*	*Excess of Collapse Load over Load to Cause First Yield (per cent)*
F8H	Warren	62·5	7·2	5·5	31
F4H	Warren	62·5	8·2	6·0	36
F2H	Warren	100	6·8	6·1	11
F3H	Warren	150	5·2	4·8	9
F7H	Triangular	62·5	16·6	—	—
F9H	Triangular	100	12·9	10·5	23
F5H	Triangular	150	6·6	5·9	12

These figures indicated that for L/r greater than about 100, the reserve of strength beyond first yield was not more than about 12 per cent though, of course, the ratio of collapse load to load causing first yield was a function of other variables such as crookedness and end eccentricity, besides L/r. The above ratios were in general agreement with values Mr. Gregory had obtained. He had been attempting to find the variation of the above ratio as a function of L/r and to use it as a design criterion for struts in the intermediate stiffness range. First yield was predicted so easily by the Southwell Plot on strains.

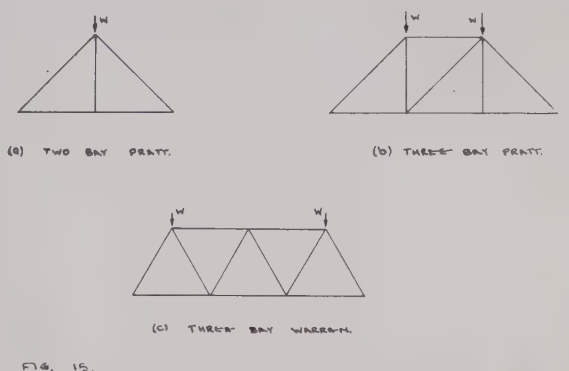

FIG. 15.

Fig. 15

THE AUTHOR in reply, stated that since the paper was submitted for publication, he had carried out tests on a further six frames made of the same material as those reported. The frames of this second series were of three types shown in Fig. 15, and details were given in Table 5.

TABLE 5

Code No.	*Type*	*Length of Compression Member*	*L/r*	*Type of Collapse*
F25H	Two Bay Pratt	15 ins.	62·5	A
F26H	Two Bay Pratt	24 ins.	100	A
F27H	Two Bay Pratt	36 ins.	150	C
F29H	Three Bay Pratt	24 ins.	100	A
F30H	Three Bay Pratt	36 ins.	150	B
F31H	Three Bay Warren	24 ins.	100	B

The deflection curves were similar to the previous set of seven and confirmed the conclusions already derived. The results were summarised in Table 6 and could be added to the points already plotted in Fig. 14.

TABLE 6

Frame No.	*Wy*	*Wc*	*Wf*	*We*	*Excess of Wf over We %*
F25H	13·9	56·4	13·5	12·8	5
F26H	14·0	27·0	12·3	11·5	7
F27H	14·0	9·7	7·4	7·3	2
F29H	7·0	35·0	6·7	5·9	16
F30H	6·9	6·2	3·9	3·9	0
F31H	8·5	14·3	6·5	4·25	53

Results were now available for about twenty trusses fabricated from rolled steel sections and covered five different types. While it was not suggested that this was comprehensive enough, it did lend confidence to the interpretation of results from small model tests. As Mr. Gregory pointed out a great many more results

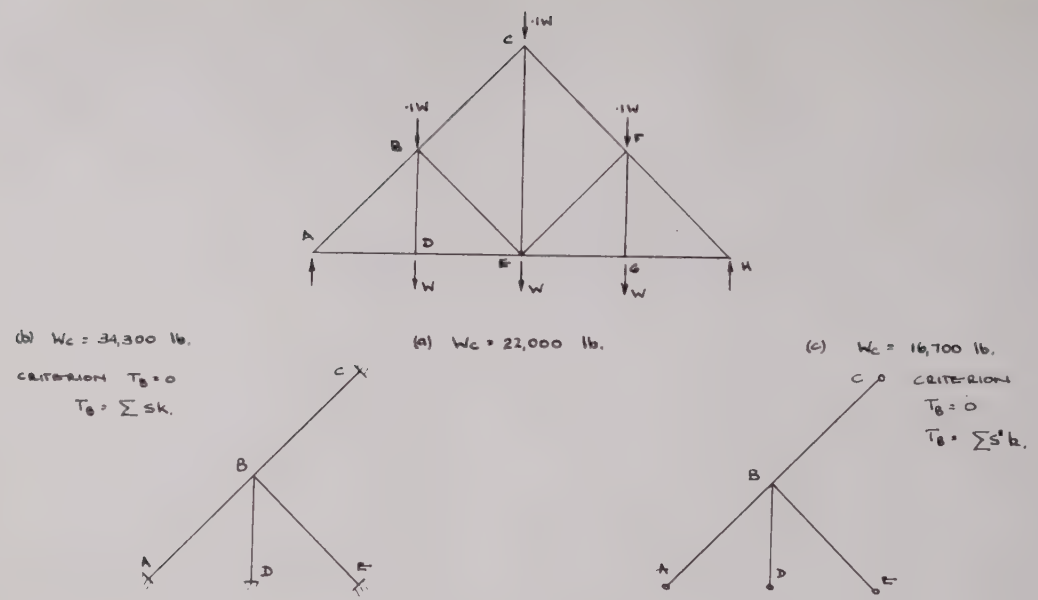

Fig. 16

of practical structures were required, and as he suggested, through the use of the Southwell Plot on the deflections, values could be obtained of the critical load of the framework. A comparison of the calculated and experimental critical loads appeared in Table 7. Unfortunately it did not lend itself to the analysis of some of the frames of stocky members. In those cases, the total deflections up to the elastic limit were so small that the deflections, read only to the nearest 0·001 in., gave too much scatter to the points. As a reliable straight line could not be drawn through them, the figures had been omitted from the table.

TABLE 7

Frame No.	*Wc (calc.)*	*Wc (expt)*	*We (expt)*	*We (calc.)*
F8H	40·5	—	5·5	5·7
F4H	41·5	38·0	6·0	5·3
F2H	15·9	16·2	6·1	5·1
F3H	7·1	5·8	4·8	5·3
F7H	50·0	55·0	16·0	11·2
F9H	14·0	13·5	10·5	10·9
F5H	8·8	8·4	5·9	6·0
F25H	56·4	—	12·8	11·5
F26H	27·0	24·0	11·5	9·7
F27H	9·7	10·3	7·3	6·75
F29H	25·0	23·8	5·9	5·5
F30H	6·2	5·3	3·9	3·25
F31H	14·1	14·3	8·45	9·0

Mr. Gregory had compared the collapse load and the first yield load of the published tests. To his Table 4 could be added the new results (Table 6, column 6). Considering the results as a whole, the excess of collapse load over first yield load was not great, even for low slenderness ratios and led to the consideration of the first yield load rather than the collapse load as the basis of analysis. The calculation of that was simple. For the tests reported, the end eccentricities of the collapsing column had been calculated under various loads so that the deflection at the centre could be found accurately. The load in the column necessary to cause yielding could be obtained from the formula

$$P = \frac{f_y}{\frac{1}{A} + \frac{1}{Z}\left[\Delta + (a + e)\right]}$$

These results were shown in Table 7, column 5 and compared favourably with experimental values.

For design purposes, the assumption of a suitable initial eccentricity was necessary if this value was not known. When a constant value of " e " was assumed throughout the loading range, the implication was that the moments at the end of the column increased linearly with load. This was known to be untrue, due to the variation in stiffness and carry-over factors for the members under changing axial load. However, for trusses with stocky members the departure from linearity was not great and it was reasonable also to apply this assumption to members in the intermediate range.

He agreed the value of initial eccentricity was only to be obtained by analysing practical structures. It was most conveniently expressed however in terms of a function of l_{ry}/∂_y as was done for the Perry-Robertson formula. Then the calculations followed the same form as that advocated by B.S.449, but guidance was given on the selection of the free length of the column. This could be obtained directly from the critical load.

It was necessary to do it rapidly and a useful approximation had been offered by Dr. N. W. Murray[11], who suggested that the joint with the lower stiffness at the end of the most highly loaded compression member, i.e. the member in which P/P_E was greatest, be selected as the critical one, and the far ends of the members framed into this be considered in turn as pinned and completely fixed. The average of these two critical loads gave a close approximation to its true value. The example Mr. Gregory had given would be appropriate. The truss of Fig. 16 (a) had been reduced to two frames, Fig. 16(b) and Fig. 16(c) and it would be seen that the average of these two critical loads was close to its accurate value.

The great disadvantage in selecting the yield load as the design criterion was its dependence upon the value of initial eccentricity. As could be seen from Figs. 7-13, the effect of moving the elastic stability line to the left or right did not change the value of W_g greatly, and hence W_f was nearly constant. However the load of first yield would change by up to 30 per cent and was much more susceptible to the assumption of the correct value of initial eccentricity.

Because of this, it was probably more logical to find the collapse load directly rather than base its determination on the increase over the yield load as Mr. Gregory suggested.

The Author thanked Mr. Gregory for an interesting discussion.

Reference

11. Murray, N. W., Chapter 8, p. 409 of "*Hyperstatic Structures*," by J. A. L. Matheson, Butterworth, 1959.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 28th April, 1960, at 5 p.m. Mr. Lewis E. Kent, B.Sc., M.I.Struct.E., M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

Ashmead, Martin Paul, of Bristol.
Betts, Robert Frederic, of Wembley, Middlesex.
Changeur, Robin Peter, of Edgware, Middlesex.
Firebrace, Andrew William, of Twickenham, Middlesex.
Graham, Arthur March, of Dunedin, New Zealand.
Hughes, Peter James, of London.
Kenley, Tom Wedgwood, of Workington, Cumberland.
Lau, Francis Shiu Kwan, of Hong Kong.
McKenzie, Ian Bruce, of Marton, New Zealand.
Newton, Alfred John, of Salisbury, Southern Rhodesia.
Powers, Alan George, of Birmingham.
Venkateswaran, Kasi Srinivasan, of Madras, India.
Verco, Alan Walter, of London.
Yam Chung Pong, Lloyd, of Hong Kong.

Graduates

Adenuga, Julius Adepoju, of London.
Al-Kotani, Nashat, of London.
Bailey, Landon, of Sidcup, Kent.
Basu, Sunandan Narayan, B.Eng., of Calcutta, India.
Blake, Ronald James, of Harrow, Middlesex.
Bobisch, Gerhard, of Durban, South Africa.
Burnley, Derek Hewitt, of Darlington, Co. Durham.
Butler, Desmond James, B.Sc., of London.
Chapman, John Michael George, of Haywards Heath, Sussex.
Clarkson, Reginald George, of Harlow, Essex.
Davies, Peter William, A.M.I.Mun.E., of Purley, Surrey.
Davis, Henry James, of London.
Dutta, Susanta Kumar, B.Eng., of London.
Ecob, John Russell, of Yagoona, New South Wales, Australia.
Finch, Victor Leopold, of Copthorne, Sussex.
Ghosh, Bikash Kumar, B.Sc., of London.
Gifford, Robert Charles, of West Byfleet, Surrey.
Gupta, Surya Prakash, B.E., of Jodhpur, India.
Haddock, Geoffrey, of Weaverham, Cheshire.
Hewson, James, of Stockton-on-Tees, Co. Durham.
Hilson, Barry Oliver, B.Sc., of Brighton, Sussex.
Hocken, Alan, of London.
Holman, Charles Edward, of Ontario, Canada.
Hughes, Windsor Griffith, of Maesteg, Glamorganshire, South Wales.
Jack, William Sommerville, of Glasgow.
Kar, Debabrata, B.Eng., of Burnpur, Dt. Burdwan, West Bengal, India.
Korevaar, Willem Huibertus, of Johannesburg, South Africa.
Ladduwahetty, Neville Sathyananda, B.Sc., of London.
Law, Matthew Kim Teng, B.Sc., of London.
Letman, Bernard, of Johannesburg, South Africa.
Levy, John, of Johannesburg, South Africa.
Lyon, David Foster, of Preston, Lancs.
Marghetis, Christor Constantin of Alexandria, Egypt.
Masters, Michael Andrew, of Bourne End, Bucks.
Mukhopadhyay, Shankar, B.Sc., of London.
Nerurkar, Anil Madhav, B.E., of Bombay, India.
Nyland, Paul, of Padiham, Lancs.
Oubridge, Denis Brian, of New Addington, Croydon, Surrey.
Parikh, Punamchand Shankerlal, of Bombay, India.
Parker, Joseph Martin, of London.
Patel, Kanubhai Ambalal, of Baroda, India.
Pearson, John Keith, of Berkhamsted, Herts.
Pitchers, John, of London.
Purkayastha, Ramapada Kar, B.Sc., of London.
Reynolds, Rodney Alan, of Barking, Essex.
Rooney, Dermot Francis, B.E., of London.
Sattar, Abdul Jabbar Abdul, of London.
Schroeder, Bryan Payne, of Cape Town, South Africa.

Shroff, Hoshang Phiroseshaw, B.E., of Bombay, India.
Smith, John Brian, of London.
Stevens, Kenneth, of Sheffield.
Stiefel, John Emil, B.Sc., of Bexley, Kent.
Thessman, Warren Richard, B.Eng., of Wellington, New Zealand.
Tilbury, John Alan, of High Wycombe, Buckinghamshire.
Todd, Bernard, of Culcheth, Nr. Warrington, Lancashire.
Toor, Khalil-Ur-Rahman, of Wah Cantt, West Pakistan.
Turner, Frank Herbert, of Shirley, Croydon, Surrey.
Wolski, Karol, of London.

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Brown, Peter Allan, B.Sc., of Epping, Essex.
Daugherty, Frank, of Liverpool, Lancs.
Divan, Malhar Shivaram, B.E., of Ahmedabad, Bombay State.
Ducker, James, of Rotherham, Yorkshire.
Griffiths, Edward John, of Clifton, Bristol.
Hollington, Michael Robert, B.Sc., A.M.I.C.E., of London.
Jones, Chandler, B.Sc., of Edinburgh.
Loat, Reginald Dennis, of Merry Hill, Wolverhampton.
Londhe, Vishwas Dattatraya, B.E., of Bombay, India.
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Scheuer, Marius Cornelis, of Johannesburg, South Africa.
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Stuart, Allan, of Liverpool.
Towson, Valentine Geoffrey, of Cape Town, South Africa.
Trussler, Stanley, of Llanhilleth, Monmouthshire.
Ward, Albert Thomas James, of London.

Transfers

Students to Graduates

Albiez, Antony Patrick, of Crumpsall, Manchester.
Athendriou, Michael, of London.
Bailey, Brian Lawrence, of Bolton, Lancs.
Basu, Mrinalkanti, B.Sc., of London.
Bhamra, Darshan Singh, of London.
Bharadwaj, Joginder Paul, of Rajasthan, India.
Carson, Samuel William, of Salisbury, Southern Rhodesia.
Chaston, Paul Bernard, of London.
Davis, Victor Clayton Aliston, of London.
Dixon, Raymond James, of Hillingdon, Middlesex.
Duffy, Patrick, of Salisbury, Southern Rhodesia.
Fatehi, Abdulkadir Abdulhasan, of Bombay, India.
Gerrard, John Roy, of Bolton, Lancashire.
Grimshaw, John, of Bury, Lancashire.
Gyani, Kishor Ranchhodlal, of Bombay, India.
Harel, Georges Alfred Pierre, of London.
Leithead, Gordon McLean, of Montreal, Canada.
Li, Edward Shu Fun, of London.
Lim Chwee Bok, of Singapore.
McCarthy, Patrick Dennis, of Johannesburg, South Africa.
Maujean, Joseph Lucien, of Johannesburg, South Africa.
Reed, John Barton, of Dublin, Eire.
Richter, Colin Philip, of Cape Town, South Africa.
Shears, Peter John Randal, of Bradford, Yorks.
White, John Ronald, of Belfast.
Whitmore, Richard Graeme, of Auckland, New Zealand.

Students to Associate-Members

Maycroft, Douglas Harvey, D.F.C., of Wellington, New Zealand.

Graduates to Associate-Members

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Bogle, John Brian, of Cheadle, Cheshire.
Botterill, John Alfred, of Dartford, Kent.
Brunt, John David, of London.
Burchess, David Sydney, of Bromley, Kent.
Chorley, Victor Wilfred, of Transcona, Manitoba, Canada.
Clayden, Kenneth John Charles, B.Sc., A.M.I.C.E., of Broxbourne, Hertfordshire.
Fox, Dennis, B.Sc., of New Malden, Surrey.
Fozzard, John, of London.
Garrad, Reginald John, of Porthcawl, Glamorganshire.
Gill, Michael Brendan Mary, B.E., of London.
Gray, Walter Smith, of Wishaw, Lanarkshire.
Harkin, Michael Edward, of London.
Harris, George Graham, B.Sc., A.M.I.C.E., of Hale Barns, Cheshire.
Hugall, John Eric, of Enderby, Leicester.
Hurden, Michael Eric, of London.
James, Arthur Neville, of Durban, South Africa.
Lambert, Frederick Walter, of London.
Lee, John Kenneth, of Stockport, Cheshire.
Lee, Terence Godfrey, of Mitcham, Surrey.
Lyth, Philip Bernard, of Middlesborough, Yorks.
McFarlane, John, of Horley, Surrey.
Patel, Janakray Rambhai, B.E., of London.
Phillips, William, B.Sc., of North Gosforth, Newcastle upon Tyne.
Ridler, John Walter Arthur, of Surbiton, Surrey.
Robson, Ronald John, of Sutton, Surrey.
Rowbotham, Noel Gervais, of Kingston-upon-Thames.
Sharland, Brian Walter, of Johannesburg, South Africa.
Sidoryk, Tadeusz, of London.
Singh, S. Balbir, of Orissa, India.
Slack, George Edwin, of Scunthorpe, Lincolnshire.
Slaney, John Richard, B.Sc., A.M.I.C.E., of Beckenham, Kent.
Snell, Martin de Putron, of Purley, Surrey.
Staples, Graham George, of Sutton Coldfield, Warwickshire.
Tawde, Ashok Nanasaheb, B.E., of Bombay, India.
Taylor, Wilfred Samuel, E.D., A.M.I.C.E., of Martinborough, New Zealand.
Tilbury, Frank Charles, of London.
Turkington, William Kenneth Somme, of Belfast, Northern Ireland.
Twigger, Michael Arnold, of Worcester.
Vithal, Nori Panduranga, B.E., of Bombay, India.
Waite, Dennis, of Stockton-on-Tees, Co. Durham.
Whittaker, James Roy, of Stretford, Lancs.
Wong Kat Cheong, Franklin, B.Sc., Ph.D., of Hong Kong.
Wyatt, Albert Frank, of Cheam, Surrey.

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Gillespie, John Hendry Hepburn, B.Sc., A.M.I.C.E., of Newcastle upon Tyne.
Grace, John Stephenson, of Croydon, Surrey.
Kumaranayagam, Rasanayagam, B.Sc., M.I.C.E., of Colombo, Ceylon.

Moss, Thomas Reginald, of Liverpool.
Radway, Ernest Raymond, M.I.Mech.E., M.I.E.E., of Cardiff, Glamorganshire.
Tembe, Nilkanth Ramchandra, B.E., M.Sc., M.I.C.E., of Bombay, India.
Walker, Albert, A.M.I.C.E., of Newcastle upon Tyne.

Associate-Members to Retired Associate-Members

Brown, Walter Lindahl, A.M.I.Mech.E., of Woodley, Berkshire.
Heaton, Hubert, of Exeter.

OBITUARY

The Council regret to announce the deaths of Kornelis Reinhold Danhof, Henry Hart, John Thomas Mackley, and Frederick Sutton-Smith (Members).

RESIGNATION

Notification was given that the Council had accepted with regret the resignation of Robert Lucien Bourqui (Associate-Member).

JUNE MEETING

An Ordinary General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 23rd June at 5 p.m.

This meeting is for the election of members and is opened to corporate members of the Institution only.

EXAMINATIONS, JULY, 1960

The Examinations of the Institution will next be held in the United Kingdom and overseas on Tuesday and Wednesday, July 12th and 13th, 1960 (Graduateship) and Thursday and Friday, July 14th and 15th, 1960 (Associate-Membership).

PRIZES JANUARY, 1960 EXAMINATIONS

The Council have awarded the following prizes in respect of examinations held in January, 1960 :

Andrews Prize.—For the candidate who obtains the highest aggregate of marks in the Associate-Membership Examination, passing in all subjects.
M. S. Divan of Ahmedabad, India.

Husband Prize.—For the candidate who takes the whole of the Associate-Membership examination, passes in all subjects, and obtains the highest marks in the paper "Structural Engineering Design and Drawing."
J. B. Bogle of Cheadle, Cheshire.

Wallace Premium (Senior).—For the candidate who takes the whole of the Associate-Membership examination, passes in all subjects, and obtains the highest marks in the paper "Theory of Structures (Advanced)."
M. S. Divan of Ahmedabad, India.

A. E. Wynn Prize.—For the candidate who takes the whole of the Associate-Membership examination, passes in all subjects, and obtains the highest marks in the Reinforced Concrete Section 'C' of the paper "Structural Engineering Design and Drawing."
B. Newsome of Morley, Yorkshire and N. G. Rowbotham of Kingston-upon-Thames.

Graham Wood Prize.—For the candidate who obtains the highest marks in the Steel Section 'B' of the paper "Structural Engineering Design and Drawing" in the Associate-Membership examination.
J. McDonald of Cardiff.

Wallace Premium (Junior).—For the most successful candidate in the Graduateship examination, passing in all subjects.
P. S. Parikh of Bombay.

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During weekdays, Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

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Hon. Secretary : W. N. Cursiter, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Redpath Brown and Co. Ltd., P.O. Box 648, Singapore.

Book Reviews

Die Berechnung Der Zylinderschalen, by Aas-Jakobsen. (Berlin/Göttingen/Heidelberg : Springer Verlag, 1958). 9 in. × 6 in., 160 plus xii pp. 38/6d.

Whilst large span cylindrical shells can be designed by the beam theory in an elementary way, this method is not applicable to shells of short and medium spans. The solution by the various existing *exact* methods requires a considerable amount of numerical work. In order to simplify the calculations, the author has introduced what he calls a ' model ' shell which is so selected that (1) the internal forces can be obtained from tables, and (2) that the boundary conditions agree with those of the actual shell with good approximation. Each edge of the shell is considered independently of the other, i.e., it is assumed that the effect of the disturbances at one edge on the deformations is completely damped before reaching the other edge. Tables are given both for isotropic shells and for shells with transverse ribs. The forces obtained for the ' model ' shell are transferred to the actual shell. The distribution in the transverse direction is maintained, whereas in the longitudinal direction it is based on the simple beam theory.

The book also deals with the analysis of shells with variable radii and with the buckling of shells. Suggestions are made for the distribution of the reinforcement along the trajectories, and for the design of stiffening beams and frames.

A short chapter is devoted to the design of prestressed shells. Numerous examples are given to illustrate the application of the theory. A comprehensive bibliography of publications on cylindrical shells in chronological order is also included.

K.H-K.

Principles of Modern Building, Vol. 1, 3rd Edition, Department of Scientific and Industrial Research. (London : H.M.S.O., 1959). 9¾ in. × 6 in., 302 + x pp. 20s.

This is a completely re-written edition of a book published in 1938, bringing up to date all the principles of design. The volume is divided into two parts, the first dealing with the principles of the functional performance of a building as a whole and including the parts of the previous editions which have general application, leaving then specific application to walls and other vertical elements to be dealt with in Part II, which incorporates most of the remainder of the original volume. Volume II will deal similarly with their application to the horizontal elements, floors and roofs. The general properties of materials that determine the ways in which they can be used are grouped together in Part I, leaving to the later parts the more specific questions that arise in their use in the particular parts of a building.

The book concludes with a glossary, lists of British Standards and suggestions for further reading. It is intended as a supplementary volume to textbooks on building construction, and as such is a useful work of reference for students as well as for all who have to deal with building construction.

Hyperstatic Structures, Vol. I, by J. A. L. Matheson. (London : Butterworth Scientific Publications, 1959). 10 in. × 6 in., 474 + xv pp., 90s.

This is an interesting new book concerned mainly with the elastic analysis of statically-indeterminate structures. The Author's intention is to expound the basic theorems of structural analysis in this first volume, so few numerical illustrations are included ; a Volume II is to follow to demonstrate the theoretical methods by means of further examples.

The first seven chapters are entitled : " Hyperstatic Structures, Energy theorems of structural analysis, General theorems for linear elastic structures, General methods for linear hyperstatic structures, Moving loads on structures and Frames with rigid joints and arches." These are the work of Professor Matheson himself ; the final eighth and ninth chapters on stability of struts and frameworks and on matrix methods are contributed by Drs. N. W. Murray and R. K. Livesley respectively.

The treatment is in the nature of a report and exposition of the various significant developments in the subject. This approach is not particularly original, though the Author does attempt to correlate some of the theorems and methods described. Few of the theorems are presented rigorously, but then the Author may have decided to sacrifice rigour for simplicity in such an introductory work. However, the book should serve a useful purpose in the instruction of students, but it offers little contribution to the subject except, perhaps, in some parts of its presentation.

One cannot help but feel, on examining this volume, that a comprehensive and masterly statement of the dual force and displacement methods of elastic structural analysis still has to be published in book form. More rigour and scope are required in the proofs of the various theorems, which would best be explained to the reader from original sources rather than from minor publications. In several instances here the originators of new devices and theories are not properly traced and credited, which seems a pity in a survey of this sort, which should establish sources correctly, or not at all.

On the whole the book is fairly up to date, though, the examples on moment distribution in section 6.4.3. could now be solved much more directly, and there are several omissions of theory and of comment on the analysis of certain common structural types. The inclusion of the last two chapters undoubtedly strengthens the book and enables two young structural analysts to explain some of the theoretical background of their own work and that of others on structural instability and electronic computing. The only criticisms here are the neglect, in chapter eight, of the simple Merchant-Rankine method for estimating the collapse load of a framework, and, in Chapter nine, of an example of the flexibility coefficients method.

The book is very readable and the layout and production are really good, so even with the limitations mentioned above this new textbook on structural theory can be recommended for student use up to degree level.

E.L.

structural steelwork

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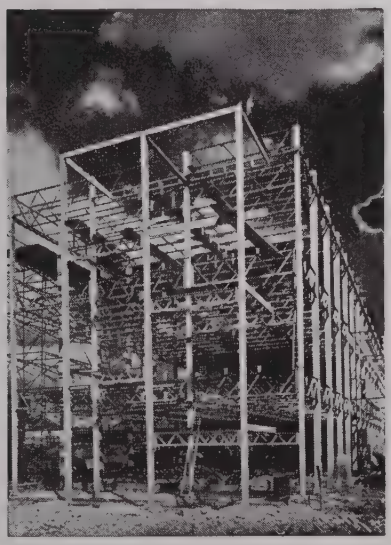

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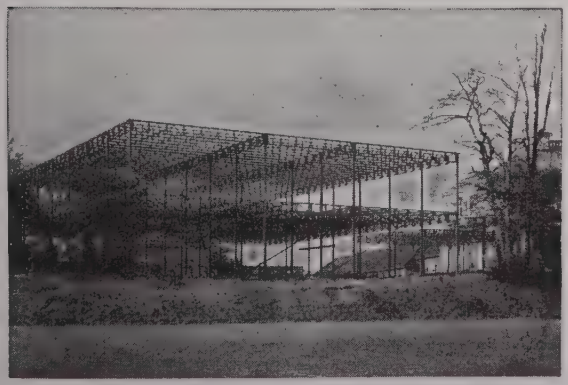

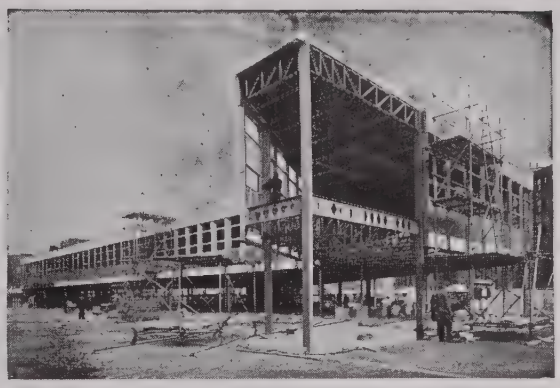

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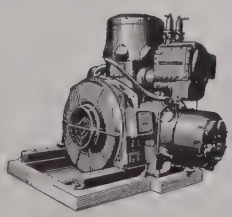

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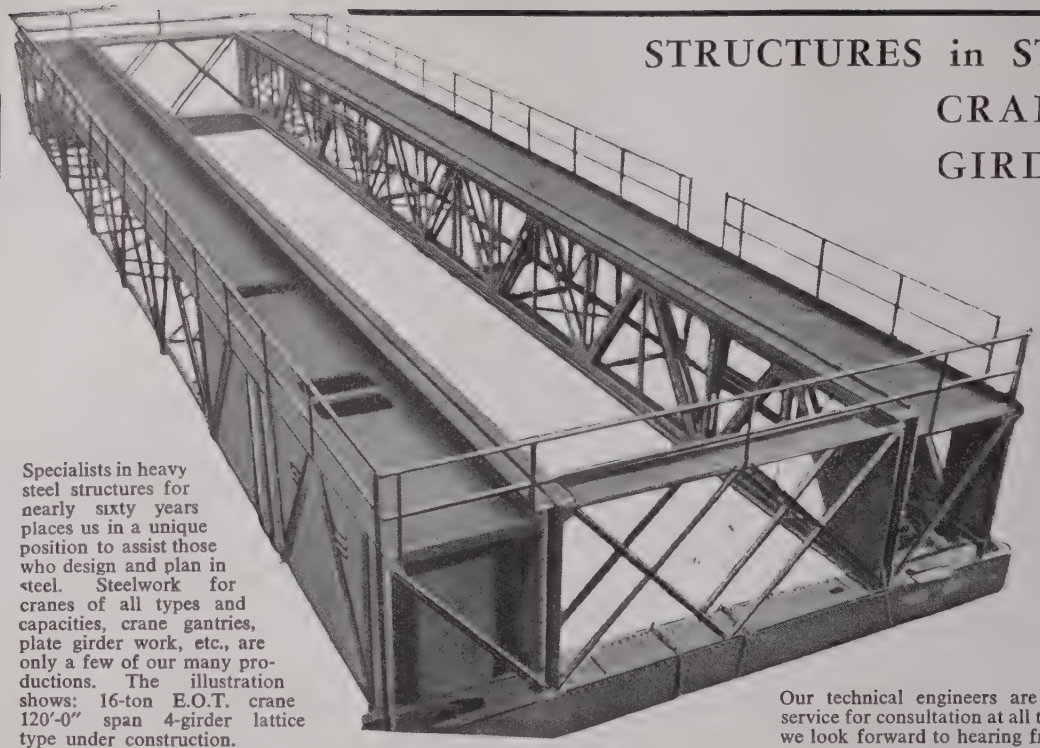

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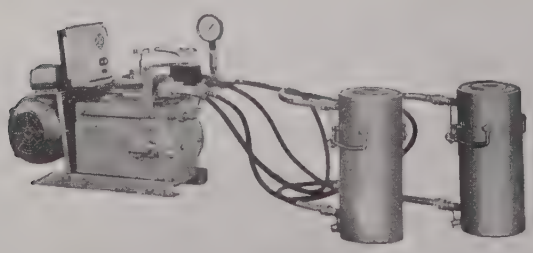

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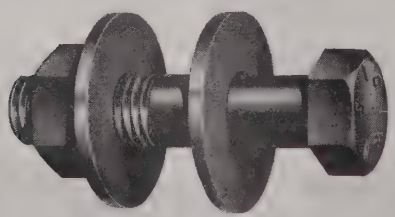

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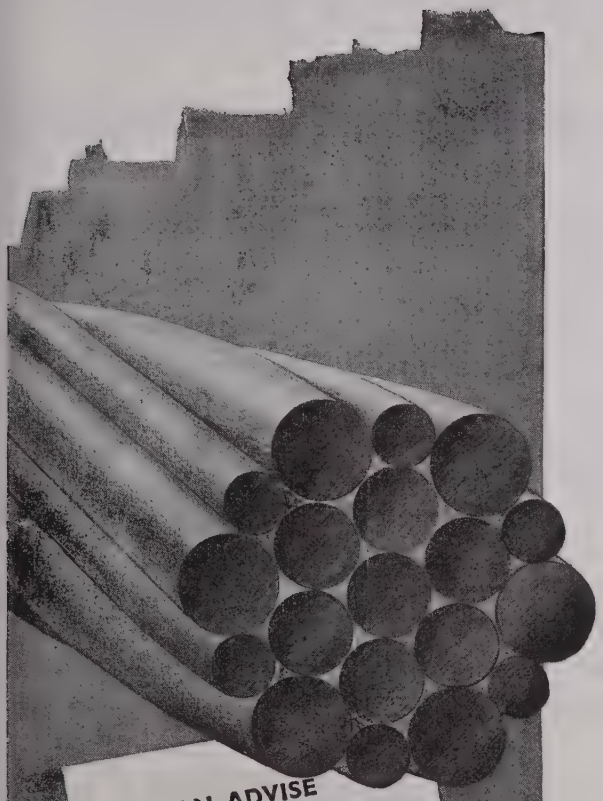

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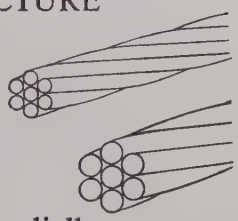

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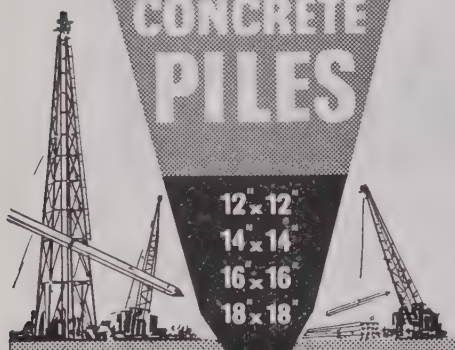

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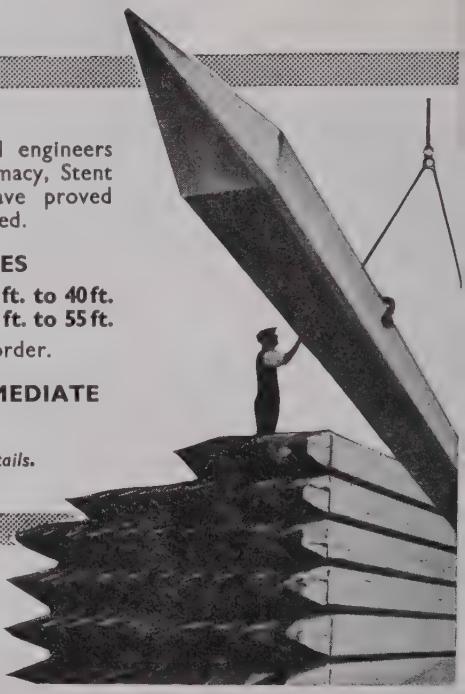

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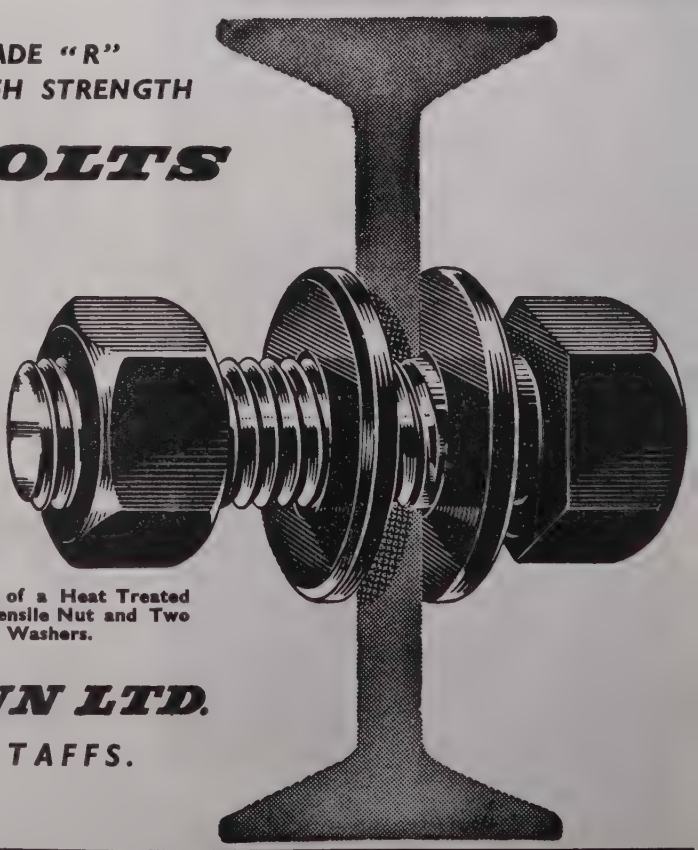

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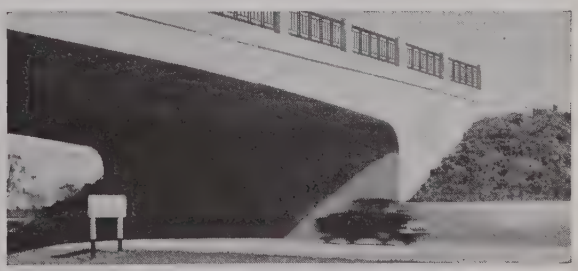

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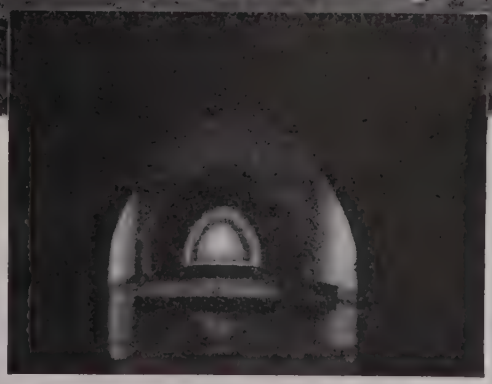

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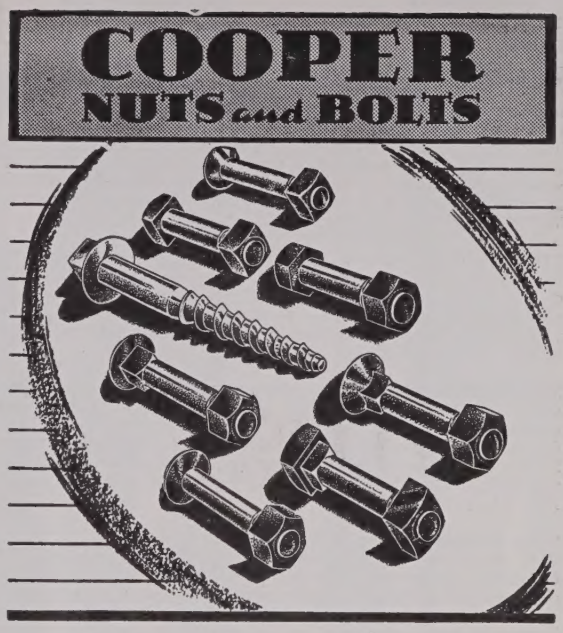

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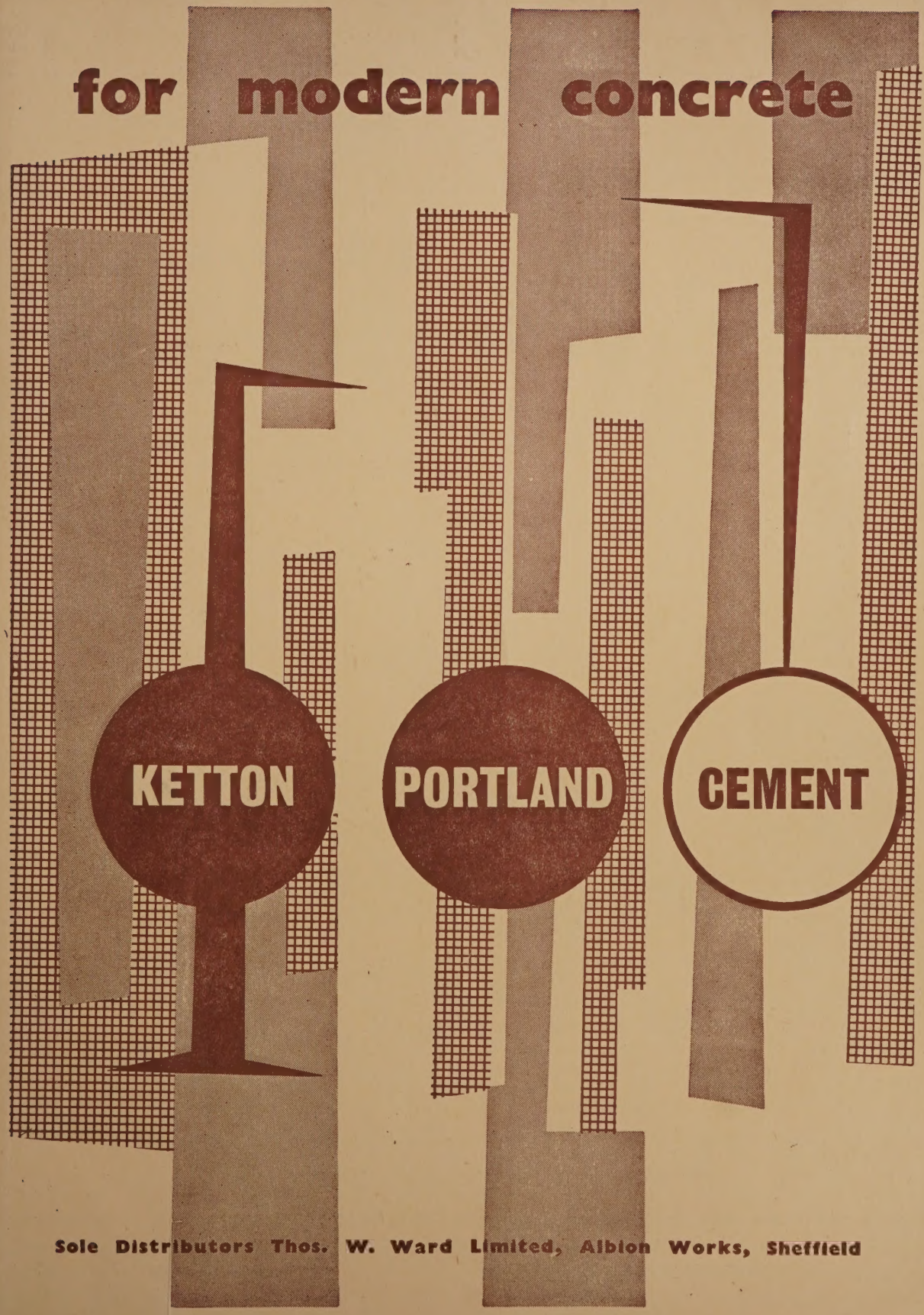
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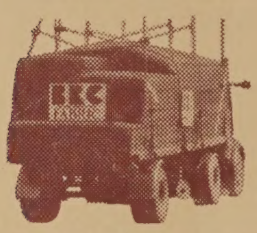

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